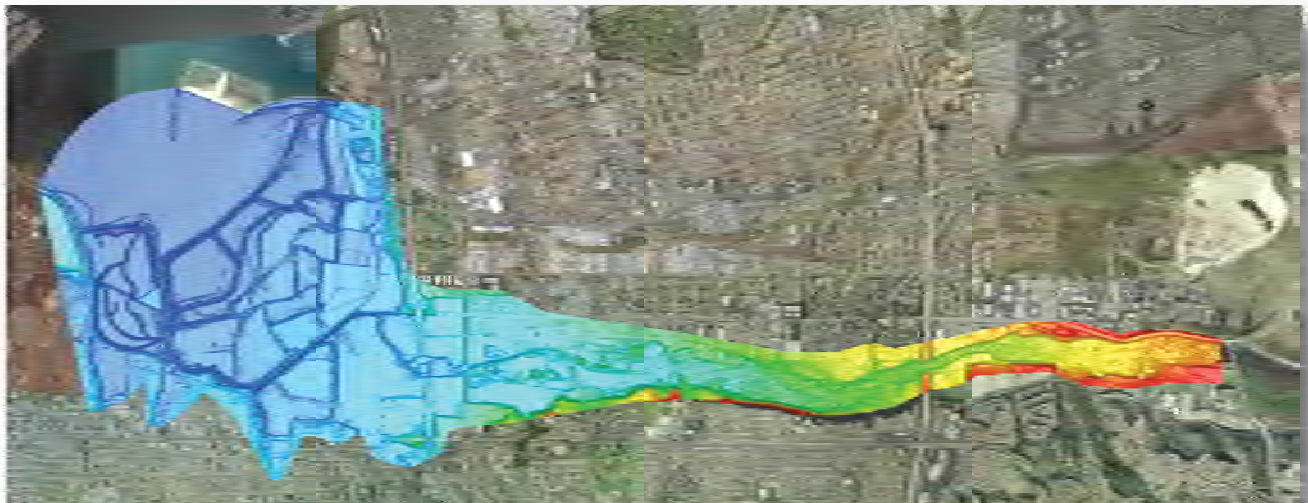# APPENDIX H
Fluvial Hydraulics Study

Otay River Estuary Restoration Project

FLUVIAL HYDRAULICS STUDY

Prepared for
Poseidon Water LLC

Prepared by
Everest International Consultants, Inc.

April 2016

EVEREST

OTAY RIVER ESTUARY RESTORATION PROJECT

FLUVIAL HYDRAULICS STUDY

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April 2016

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LIST OF ACRONYMS

%	Percent
2-D	Two-dimensional
Alt	Alternative
Ave	Avenue; Average
Blvd	Boulevard
CCC	California Coastal Commission
CDP	Coastal development permit
cfs	cubic feet per second
CO-CAT	California Climate Action Team
COPC	California Ocean Protection Council
DEM	Digital Elevation Model
Everest	Everest International Consultants, Inc.
Exist.	Existing
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
ft	feet
hr	hour
I-5	Interstate Highway 5
in	inch
km^2	square kilometer
m^3	Cubic meter
max	maximum
MHHW	mean higher high water
mi^2	square mile
MLLW	mean lower low water
MLMP	Marine Life Mitigation Program
MLW	mean low water

mm	millimeter
MTL	mean tide level
N	north
NAVD88	North American Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
NRC	National Research Council
NTDE	National Tidal Datum Epoch
ORERP	Otay River Estuary Restoration Project
ORF	Otay River Floodplain
PMP	Parametric Mean Periodic
Poseidon	Poseidon Water LLC
Refuge	South San Diego Bay National Wildlife Refuge
S	south
SLR	Sea level rise
SR	State Route
USACE	United States Army Corps of Engineers
USFWS	U.S. Fish and Wildlife Service
yd^3	Cubic Yard
yr	year

1. INTRODUCTION

1.1 BACKGROUND

Poseidon Water LLC (Poseidon) is currently constructing a desalination plant near the ocean in Carlsbad, California in the western portion of Agua Hedionda Lagoon. To obtain a coastal development permit (CDP) for the desalination plant from the California Coastal Commission (CCC), Poseidon was required to develop and implement a Marine Life Mitigation Program (MLMP). One of the components of the MLMP was the planning, design, construction, operation, management, and monitoring of a coastal wetlands restoration project that would mitigate for the impacts to marine fish associated with operation of the desalination plant. Poseidon selected the Otay River Floodplain (ORF) located within the South San Diego Bay National Wildlife Refuge (Refuge) as the site for this restoration project, which is now known as the Otay River Estuary Restoration Project (ORERP). The ORERP will involve earthwork (cut and fill) within the floodplain to create the subtidal, unvegetated intertidal (mudflat), and vegetated coastal salt marsh habitats required under the MLMP. Earthwork within a floodplain has the potential to cause significant adverse impacts to flood conditions compared to the conditions that exist at present (existing conditions). Poseidon retained Everest International Consultants (Everest) to conduct a fluvial (riverine) hydraulic study to address the potential for the ORERP to cause significant adverse impacts to flooding. The purpose, objectives, methods, results, conclusions, and recommendations of the fluvial hydraulics study are summarized in this report.

1.2 PURPOSE

The purpose of the study summarized in this report was to determine whether the proposed alternatives that comprise the ORERP would result in significant adverse impacts to flooding and to develop mitigation measures to eliminate such significant adverse impacts.

1.3 OBJECTIVES

The following objectives were identified to achieve the study purpose.

- Estimate flood water levels and extent under existing and proposed conditions.
- Evaluate the impact of future projections of sea level rise on flood water levels.
- Evaluate the impact of erosion under existing and proposed conditions.
- Estimate fluvial sediment transport potential under existing and proposed conditions.

2. EXISTING CONDITIONS DESCRIPTION

A sufficient description of both the existing and proposed restoration physical conditions was necessary to perform the flood hydraulic analysis. The types of information that are crucial to these analyses are topography and bathymetry (existing and proposed conditions), ocean water levels, and surface runoff in the rivers (floods). These data were collected from field surveys conducted for this project as well as as-built drawings and other published reports gathered from the USFWS and National Oceanic and Atmospheric Administration (NOAA).

2.1   PROJECT LOCATION

The project is located at the southern end of San Diego Bay within the Refuge, as shown in Figure 2.1. The Refuge is managed by the U.S. Fish and Wildlife Service (USFWS) and consists of the Sweetwater Marsh Unit and South San Diego Bay Unit. The refuge is located about ten miles north of the United States and Mexico border in San Diego County, California and is surrounded by the Cities of National City, Chula Vista, San Diego, Imperial Beach, and Coronado.

The ORF site is situated within the Otay River Floodplain located within the Refuge South San Diego Bay Unit, as depicted in Figure 2.2. In the figure, the approximate limits of the South San Diego Bay Unit are indicated by the orange lines. The South San Diego Bay Unit extends from the ORF through the salt ponds and into San Diego Bay. The salt ponds are a system of diked evaporations ponds that covers approximately 1,060 acres. Three of the salts ponds (Ponds 10A, 10, and 11) are the site of the Western Salt Ponds Restoration Project, as indicated by the green lines. The proposed ORERP will involve earthwork in two of the existing salt ponds, Pond 20A within the ORF and Pond 15.
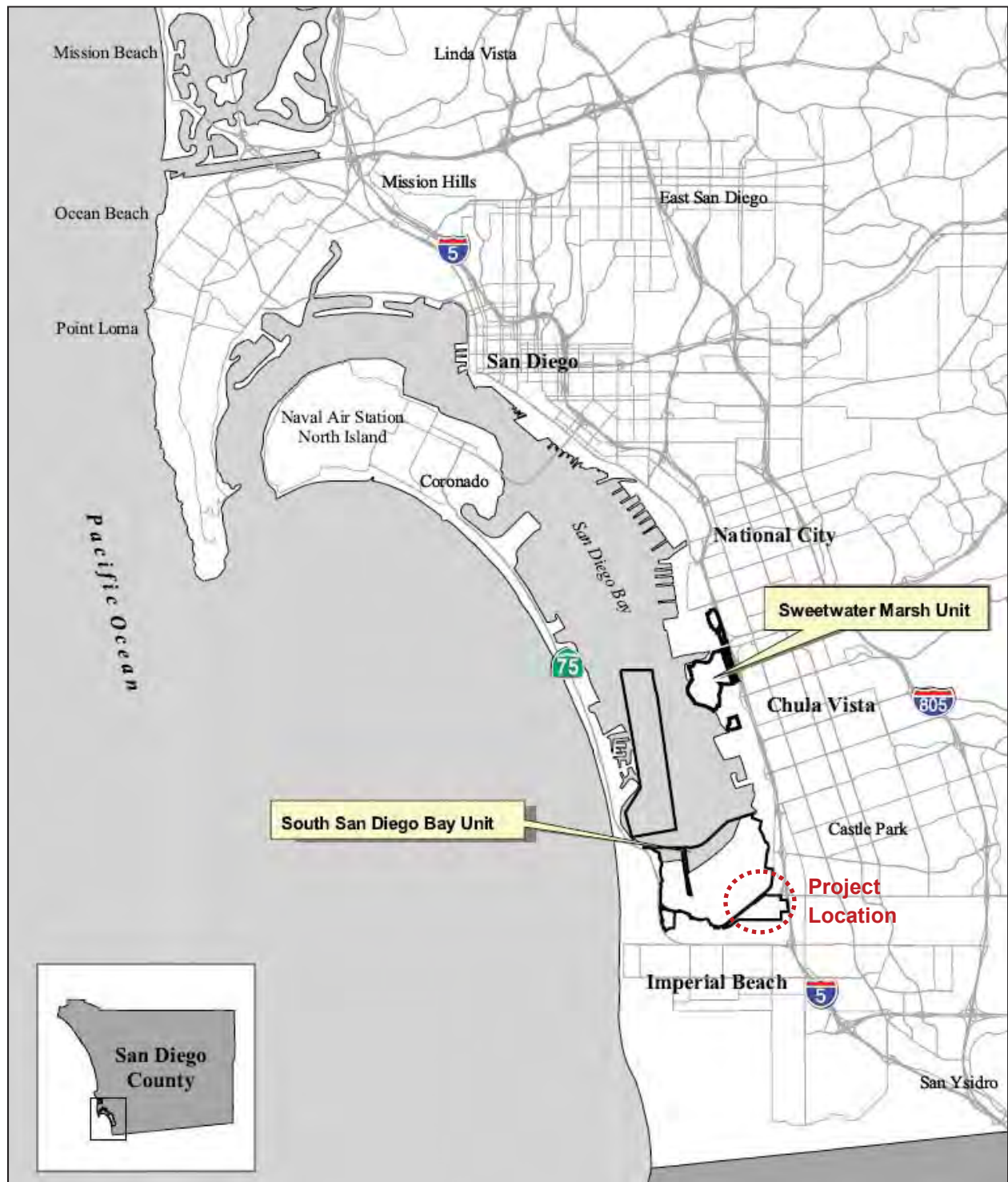
Source: USFWS 2006

Figure 2.1 Project Location and Vicinity

Figure 2.2   Proposed ORERP Area

2.2   OTAY RIVER

The Otay River originates in the Cleveland National Forest along Dulzura Creek, as shown in Figure 2.3. Tributaries include Hollenbeck Canyon Creek, Jamul Creek, and Proctor Valley Creek. Flows from the upper watershed are cutoff by two reservoirs that are a part of the City of San Diego water supply system. The Upper Otay Reservoir, which is the smaller of the two reservoirs, is located at the end of Procter Valley Creek. The upper reservoir is connected to the Lower Otay Reservoir formed by Savage Dam below the Dulzura and Jamul Creek confluence. Essentially all flows from the upper 68% of the watershed are impounded by the Lower Otay Reservoir. The upper watershed is largely comprised of undeveloped lands in unincorporated areas of San Diego County. The terrain is characterized by higher elevations and steep mountain slopes that are prone to wildfires.

The Otay River runs approximately 11 miles from Savage Dam to San Diego Bay. The river flows westward from Savage Dam through primarily undeveloped lands. The natural creek channel is transected by the South Bay Expressway (SR 125) and connected to several tributaries including Salt Creek, O'Neal Canyon Creek, Johnson Canyon, and Dennery Canyon. Downstream of the I-805, the watershed becomes urbanized and the river is heavily vegetated with sections of riprap banks. Major tributaries include Poggi Canyon Creek and Nestor Creek, which connect with the Otay River near the I-805 and I-5 bridges, respectively.

2.3   OTAY RIVER FLOODPLAIN AND ESTUARY

The Otay River conveys flows from the I-5 Bridge through the Otay River Floodplain and estuarine portion of the Otay River, as illustrated in Figure 2.4. The river channel extends northwest until reaching the salt ponds between Ponds 50 and 51 and then turns westward along the salt pond perimeter and then southwest along the Bayshore Bikeway (adjacent to Ponds 48, 22, and 20). Nestor Creek runs northward along the east edge of Pond 20A and joins the Otay River near Pond 20. The river is channelized between the salt pond dikes. After the confluence with Nestor Creek, the Otay channel is divided by a bike path bridge into two parallel segments along Pond 22. The Otay River then turns northwest beneath a second bike path bridge and then converges back into a single channel. The river continues along Pond 23 and then north along the Western Salt Pond Restoration until discharging into San Diego Bay.

The ORF and estuary area is generally flat ranging from 18 to -5 ft, NAVD88. The existing topography and bathymetry are illustrated in Figure 2.5. The existing bathymetry includes the Western Salt Ponds Restoration Project which will convert former salt ponds to approximately 230-acres of restored habitat area. The restoration site was opened tidal exchange in August 2011.

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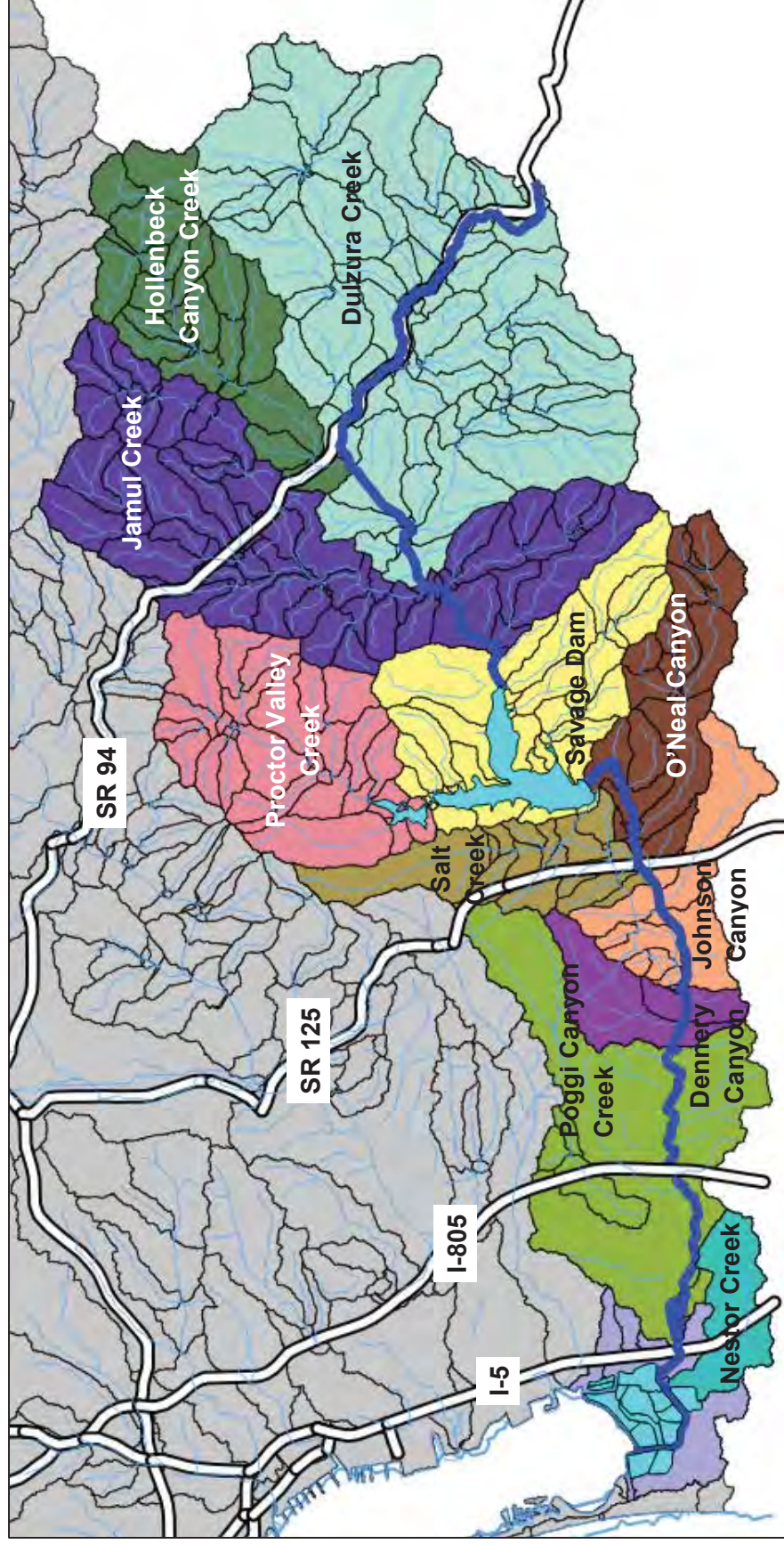
Figure 2.3   Otay River Watershed

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Figure 2.4   Otay River Floodplain

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Elevation ft, NAVD88

20
18
16
14
12
10
8
6
4
2
0
-2
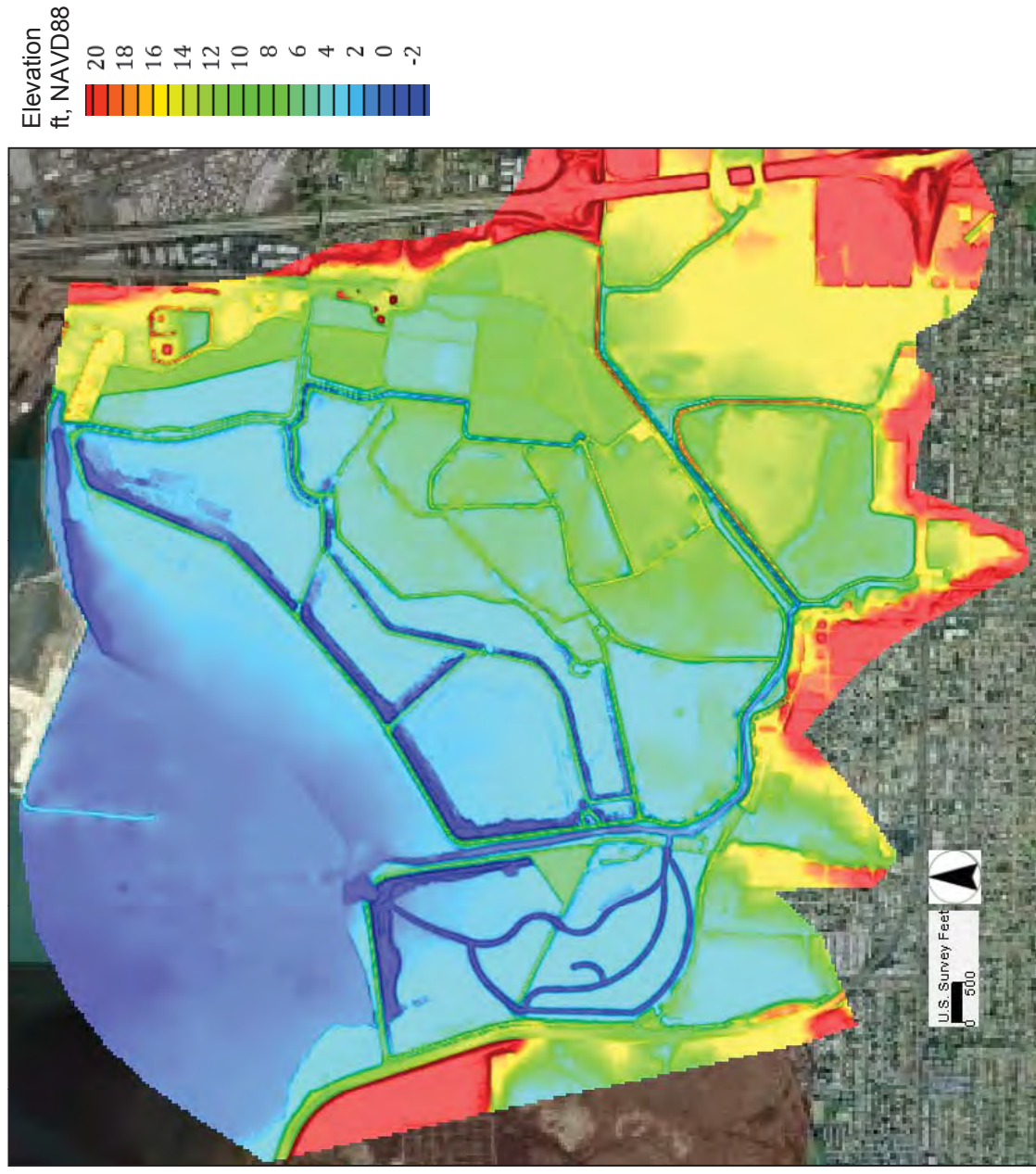
Figure 2.5 Existing Topography and Bathymetry

Everest International Consultants, Inc.

Hydraulic conditions along the Otay River are affected by a combination of tidal exchange with San Diego Bay and watershed flows from the Otay River. Tidal conditions in San Diego Bay are of the mixed, semi-diurnal type with two high and two low daily peaks. The mean tide range for San Diego Bay is 5.72 ft (NOAA 2007). Tidal influence extends from San Diego Bay towards the floodplain near Ponds 48 and 50. Fluvial flows from the Otay River pass through the ORF and can overtop the levees surrounding the salt ponds. For large floods, a portion of flood waters can be diverted through the salt ponds, filling the ponds and eventually overtopping the levees into San Diego Bay. Tide and flood water levels are discussed in greater detail in Section 4.3.

Sediment characteristics within the lower Otay River range from sandy gravel to clayey fine sand (GEOCON 1985). Boring data on sediment conditions in the floodplain area are available from GEOCON (1985), Geotechnics (2000), and USDA (2007).

3. PROPOSED PROJECT DESCRIPTION

3.1 OVERVIEW

The Otay River Estuary Restoration Project (ORERP) is located within the South San Diego Bay Unit of the San Diego Bay National Wildlife Refuge (Refuge), in San Diego County, California. Restoration activities would occur at two separate non-contiguous locations within the Refuge: (i) the Otay River Floodplain (ORF) Site and (ii) the Pond 15 Site. The approximately 79-acre ORF Site is located west of Interstate 5 (I-5) between Main Street to the north and Palm Avenue to the south. The Pond 15 Site consists of an approximately 85-acre solar salt pond located in the northeast portion of the Refuge, to the northwest of the intersection of Bay Boulevard and Palomar Street in Chula Vista.

The ORERP would involve excavation of a portion of the ORF Site and fill of the Pond 15 Site to create elevations suitable for subtidal, intertidal mudflat, intertidal coastal salt marsh, and transitional habitats as well as associated uplands. Restoration conducted in the ORF Site would be limited to the portion of the floodplain located west of the Nestor Creek, as shown in Figure 3.1 and Figure 3.3. Within this portion of the ORF Site the ground would be lowered to elevations suitable to support the target wetland habitats and wetland-associated upland habitats. In addition, the existing dike running through Pond 20A would be removed and the flood protection functionality of this feature would be replaced through construction of a levee along the southern boundary of this portion of the ORF Site. No restoration activities would be conducted in the former agricultural areas east of Nestor Creek, but this area would be available and could be used for staging construction activities. In addition to the work in the ORF Fill and Pond 15 Site, a portion of the existing dike between Salt Ponds 22 and 23 would be raised two feet to offset potential project-induced flood impacts at Bayside Park in Imperial Beach. An analysis was conducted to identify potential mitigation measures to offset these potential project-induced flood impacts. This analysis is provided in Appendix A. Besides earthwork, the restoration project might include slope armoring (e.g., riprap) to protect the Bayshore Bikeway Bridge. A conceptual design for this slope protection is provided in Appendix B.

Two restoration alternatives were developed for the project. The first alternative is known as the Intertidal Alternative and the second alternative is known as the Subtidal Alternative. Both alternatives occupy the same footprint and achieve between 19.2 to 20.9 acres of net restoration in the ORF Site. In comparison to the Intertidal Alternative, the Subtidal Alternative would provide a deeper open water channel/area within the ORF Site. In addition, the Subtidal Alternative would provide higher elevations within the Pond 15 Site due to the additional fill material associated with the deeper excavation within the ORF Site. The two restoration alternatives are described below.

3.2 INTERTIDAL ALTERNATIVE

3.2.1 Habitat Distribution

The Intertidal Alternative is composed of approximately 20% intertidal mudflat and 80% intertidal salt marsh as shown in Figures 3.1 and 3.2. Under this alternative no subtidal habitat is proposed within the ORF Site. This alternative would involve excavation and grading of the ORF Site (Figure 3.1) to create approximately 37.0 acres of tidally influenced habitats, consisting of 7.3 acres of intertidal mudflat and 29.6 acres of intertidal coastal salt marsh habitat. The Pond 15 Site (Figure 3.2) would be filled to create approximately 83 acres of tidally influenced habitats composed of 9.8 acres of subtidal habitat, 18.1 acres of intertidal mudflat, and 55.3 acres of intertidal salt marsh below +6.6 ft NAVD88. Both the ORF Site and the Pond 15 Site would be planted with a mix of native wetland vegetation that would mature into low marsh, mid marsh, and high marsh vegetative communities. The intertidal areas and unvegetated mudflat would provide foraging habitat for adult and juvenile fish.

3.2.2 Mitigation Credit

The Intertidal Alternative would provide adequate mitigation credit to meet the MLMP requirement of 66.4 acres. With a functional lift of 0.75 in the Pond 15 Site, this alternative would provide 60.5 acres of mitigation credit within the Pond 15 Site. When impacts to existing wetlands are subtracted, the Intertidal Alternative would provide 59.4 acres of mitigation credit within the Pond 15 Site. After calculating the impacts to existing wetlands, approximately 20.9 acres of mitigation credit would be provided under this alternative within the ORF Site. Therefore, the Intertidal Alternative would provide a total of approximately 80.3 acres of wetlands for mitigation credit or about 13.8 more acres than the 66.4 acres required by the MLMP.

3.2.3 Earthwork

The Intertidal Alternative would involve the excavation of 320,000 cubic yards (yd^3) of soil from the ORF Site. Approximately 21,100 yd^3 of this soil, assuming it is suitable, would be used to construct the levee along the southern boundary this portion of the ORF Site. Approximately 295,179 yd^3 of the excavated soil would be transported to and placed within the Pond 15 Site as beneficial reuse to raise the ground to elevations suitable to create tidal wetlands and associated upland habitats. Any remaining material would be transported to the portion of the ORF Site east of Nestor Creek where it would be stockpiled for future use by the U.S. Fish and Wildlife Service (FWS). If suitable material cannot be found onsite to construct the levee along the southern boundary, then 21,100 yd^3 of suitable material would be imported and the 21,100 yd^3 of soil excavated from the ORF Site would be transported to the stockpile location.

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Source: KTUA

Figure 3.1   Intertidal Alternative – ORF Site

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Source: KTUA

Figure 3.2   Intertidal Alternative – Pond 15 Site

3.3 SUBTIDAL ALTERNATIVE

3.3.1 Habitat Distribution

The Subtidal Alternative is composed of approximately 19% subtidal, 18% intertidal mudflat, and 63% intertidal salt marsh, as shown in Figures 3.3 and 3.4. This alternative would involve excavation and grading of the ORF Site (Figure 3.3) to create approximately 37.0 acres of tidally influenced habitats consisting of 5.2 acres of subtidal habitat, 8.1 acres of intertidal mudflat, and 23.5 acres of intertidal coastal salt marsh habitat. Filling of the Pond 15 Site (Figure 3.4) would result in 79.9 acres of tidally influenced habitats composed of 9.2 acres of subtidal habitat, 16.1 acres of intertidal mudflat, and 56.5 acres of intertidal salt marsh below +6.6 ft NAVD88. Both the ORF Site and the Pond 15 Site would be planted with a mix of native wetland vegetation that would mature into low marsh, mid marsh, and high marsh vegetative communities. The subtidal areas would provide spawning and foraging habitat, and the unvegetated mudflat would provide foraging habitat for adult and juvenile fish.

3.3.2 Mitigation Credit

The Subtidal Alternative would provide adequate mitigation credit to meet the MLMP requirement of 66.4 acres. With a functional lift of 0.75 in the Pond 15 Site, the Subtidal Alternative would provide 59.9 acres of mitigation credit within the Pond 15 Site. When impacts to existing wetlands are subtracted, the Subtidal Alternative would provide 57.5 acres of mitigation credit within the Pond 15 Site. Approximately 20.9 acres of mitigation credit would be provided under this alternative within the ORF Site. Therefore, the Subtidal Alternative would provide a total of approximately 78.3 acres of wetlands for mitigation credit or about 11.9 more acres than the 66.4 required by the MLMP.

3.3.3 Earthwork

The Subtidal Alternative would involve the excavation of 370,000 cubic yards (yd^3) of soil from the ORF Site. Approximately 21,600 yd^3 of this soil, assuming it is suitable, would be used to construct the levee along the southern boundary this portion of the ORF Site. Approximately 312,000 yd^3 of the excavated soil would be transported to and placed within the Pond 15 Site as beneficial reuse to raise the ground to elevations suitable to create tidal wetlands and associated upland habitats. Any remaining material would be transported to the portion of the ORF Site east of Nestor Creek where it would be stockpiled for future use by the USFWS. If suitable material cannot be found onsite to construct the levee along the southern boundary then 21,600 yd^3 of suitable material would be imported and the 21,600 yd^3 of soil excavated from the ORF Site would be transported to the stockpile location.

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Source: KTUA

Figure 3.3   Subtidal Alternative – ORF Site

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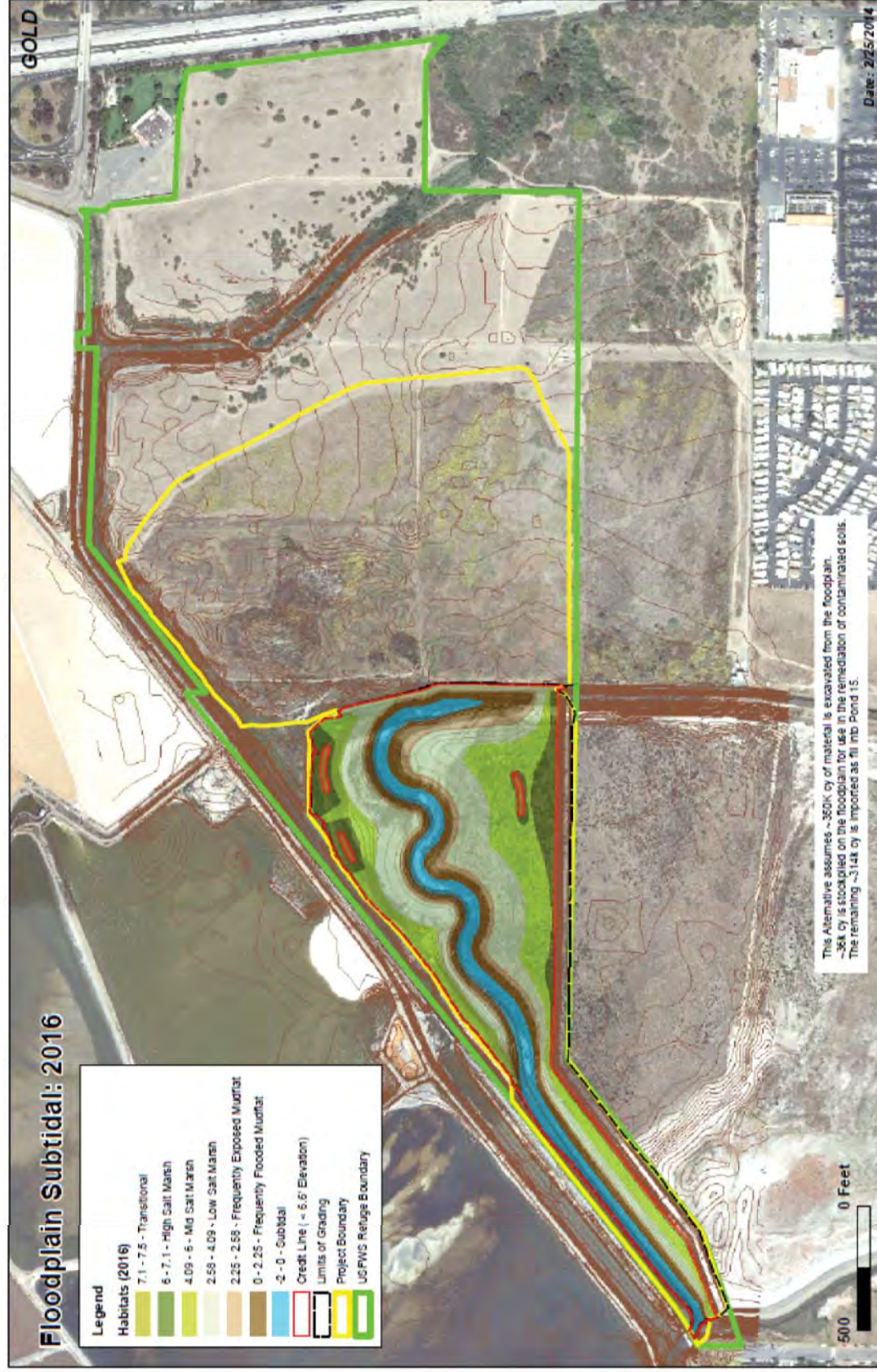
Source: KTUA

Figure 3.4   Subtidal Alternative – Pond 15 Site

4. STUDY APPROACH

4.1 OVERVIEW

Flood and erosion impacts were evaluated by comparing hydrodynamics under existing and proposed conditions. Water levels and velocities were assessed using a numerical model to simulate tidal and fluvial conditions in the project area. The flood impact analyses focused on changes to flood water levels associated with proposed conditions. The erosion impact analysis evaluated project-induced velocity changes as a surrogate for erosion (scour) potential.

The two-dimensional (2-D) hydrodynamic model TUFLOW was selected for the fluvial hydraulic analysis because the model accounts for all the necessary analysis components – tidal fluctuations, flood flows, grading changes, water control structures (e.g., open channels, culverts, pipes, and weirs), levees, and salt pond configurations. TUFLOW is a finite difference model designed for tidal and fluvial hydraulics in rivers, estuaries, coastal bays, floodplains, and urban areas.

The fluvial sedimentation analysis was conducted to identify potential impacts regarding fluvial sediment delivery and sedimentation associated with the proposed project. Analytical methods and existing data were used to estimate fluvial sediment loads from the watershed, which were then used to estimate potential sedimentation of the proposed wetland.

4.2 MODEL SETUP

Model grids were developed for existing conditions and the two proposed alternatives. The model grid for existing conditions was created first. Model grids for the two proposed alternatives were then generated by modifying the existing conditions grid based on the proposed grading of the alternatives. Hence, all three grids have the same spatial extent and are the same outside of the proposed project area, including the Otay River and Western Salt Ponds (Ponds 10A, 10, and 11).

The model domain for existing conditions is shown in Figure 4.1. The model grid consists of a 40 ft by 40 ft grid. The model domain extends from upstream boundary of Otay River at Otay Valley Road to San Diego Bay at the downstream boundary. The model domain includes bridges and culverts as well as input locations for Poggi Canyon and Nestor Creek flows. Details for the model development are provided below.

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Aerial Image: Google Earth Pro
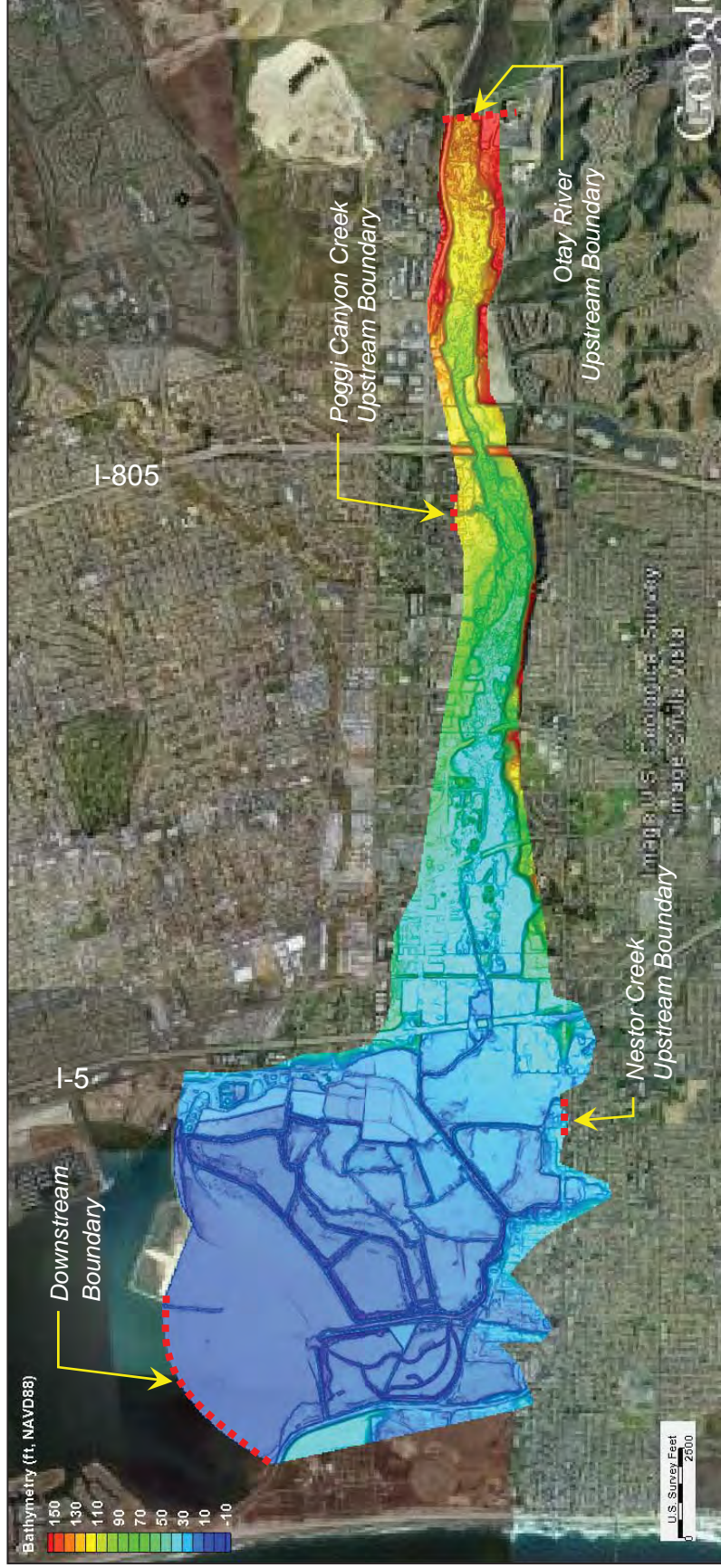
Figure 4.1 Model Domain for Existing Conditions

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4.2

4.2.1 Existing Conditions

The lower portion (floodplain) of the model domain for existing conditions downstream of the I-5 Bridge is shown in Figure 4.2. The model floodplain was selected to include higher elevations sufficient for anticipated flood water levels. Bathymetry and topography for existing conditions were compiled from various sources as summarized in Table 4.1. Elevations for the Otay River were based on a 2011 survey. In the floodplain area and salt ponds, elevations were obtained from a May 2000 survey. This data was updated based on additional surveys in 2011 and 2012. Bathymetry for Ponds 10A, 10, and 11 was based on the design and as-built surveys from the Western Salt Pond Restoration Project, which was constructed in 2011. Bathymetry for Salt Ponds 12 – 15 was surveyed in 2012. Additional bathymetry data was obtained from the NOAA DEM database.

Table 4.1   Bathymetry and Topography Data Sources

AREA	DATA	SOURCE
Otay River	Otay River survey	NWS 2011
Salt Ponds	Sweetwater Marsh and South San Diego Bay Units CCP Topographic Survey	DU 2001
Western Salt Ponds	Western Salt Pond Restoration Project	2011 Survey
Salt Ponds 12 – 15	Salt Ponds 12 -15 Survey	NWS 2012
Other	NOAA DEM Database	NOAA

The model also incorporates several bridges and culverts in the ORF (ORF). Flows beneath the I-5 Bridge and bike path crossings are obstructed from the bridge piers and deck which reduce the cross sectional area for flow, and were simulated as flow constrictions with a blockage factor. Blockage of the I-5 Bridge was estimated based on as-built drawings of the bridge. For the two bike path crossings, the blockages were approximated based on visual observations of the bridges. There are culverts located between Ponds 10 and 10A and along the river channel downstream of the bike path crossings. These culverts were simulated as a one-dimensional structure defined based on the culvert diameter and invert elevation.

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Elevation
ft, NAVD88

20
18
16
14
12
10
8
6
4
2
0
-2
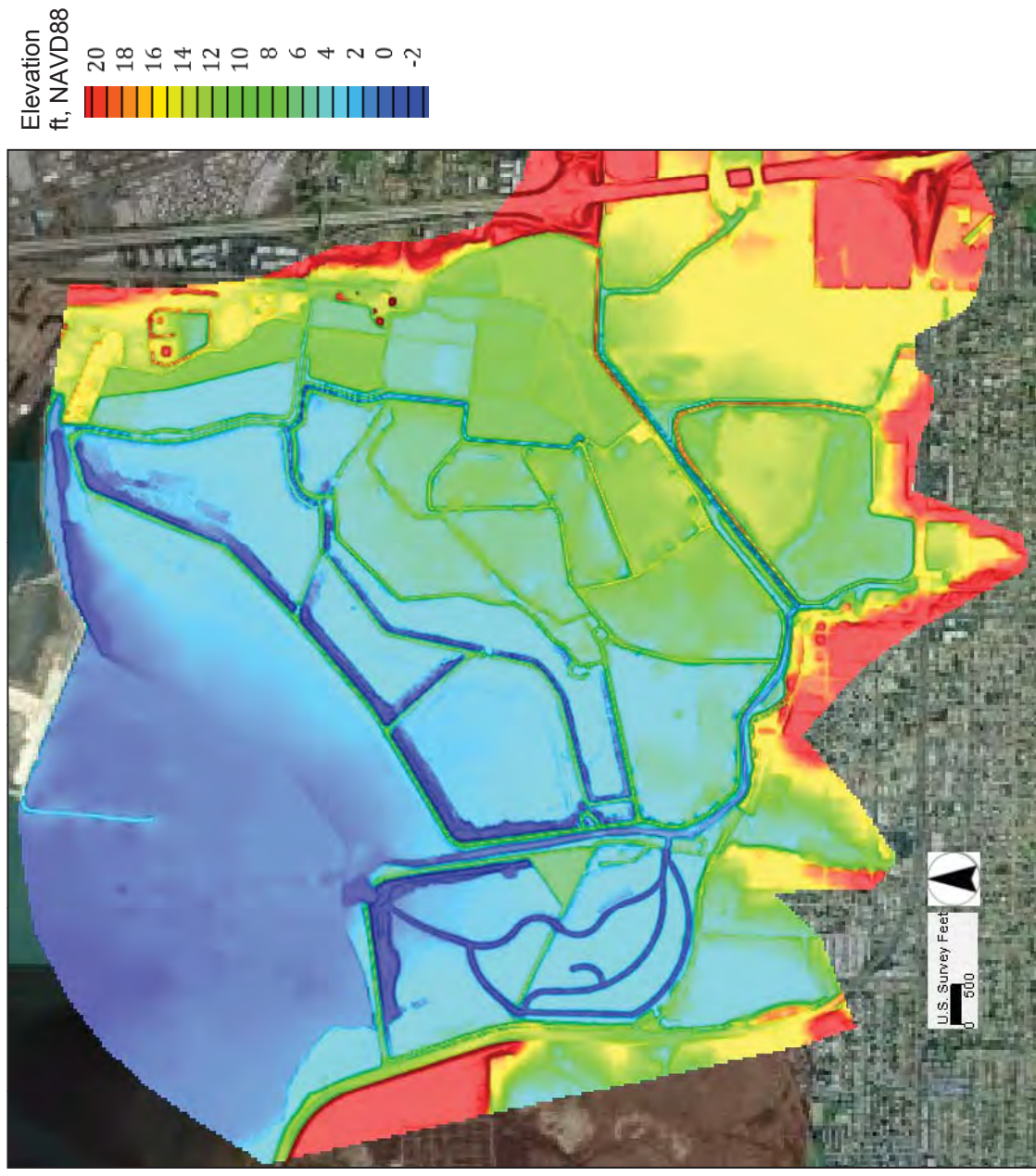
Figure 4.2 Model Bathymetry for Existing Conditions

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4.2.2   Intertidal Alternative

The model grid for the Intertidal Alternative was created by modifying the existing conditions bathymetry based on the proposed grading and sediment placement for this alternative as described previously in Section 3.2. The model bathymetry for the Intertidal Alternative and the change in bathymetry from existing condition are provided in Figure 4.3. In the figure, the left panel shows the bathymetry under the Intertidal Alternative. Modifications to the bathymetry include grading within the ORF, stockpiles within the ORF, and Pond 15. The bathymetry changes from existing conditions are shown in the right panel. In the figure, locations with reductions in elevation (cut) are shown in blue while locations with increases in elevation (fill) are shown in yellow, orange, and red, which are primarily located in Pond 15.

4.2.3   Subtidal Alternative

For the Subtidal Alternative, the existing model grid bathymetry was modified based on the proposed grading and sediment placement described previously in Section 3.3. The model bathymetry and the change in bathymetry for the Subtidal Alternative are shown in Figure 4.4. The bathymetry under the Subtidal Alternative is depicted in the left panel and the changes in bathymetry from existing conditions are provided in the right panel. Under the Subtidal Alternative, a subtidal channel and associated intertidal habitats will be constructed within the portion of the ORF west of Nestor Creek and sediment will be placed Pond 15 as well as within the two stockpile areas located within the portion of the ORF east of Nestor Creek.

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Elevation
ft, NAVD88

20
18
16
14
12
10
8
6
4
2
0
-2

Cut and Fill (ft)

10.0
8.0
6.0
4.0
2.0
0.0
-2.0
-4.0
-6.0
-8.0
-10.0

Fill

Cut
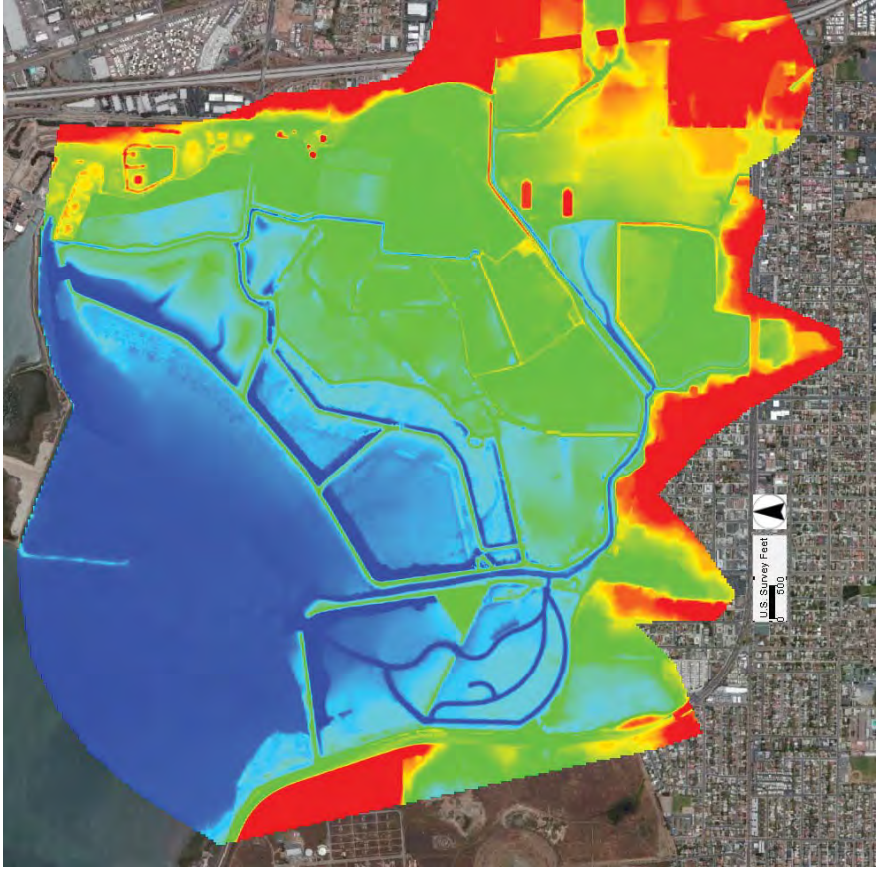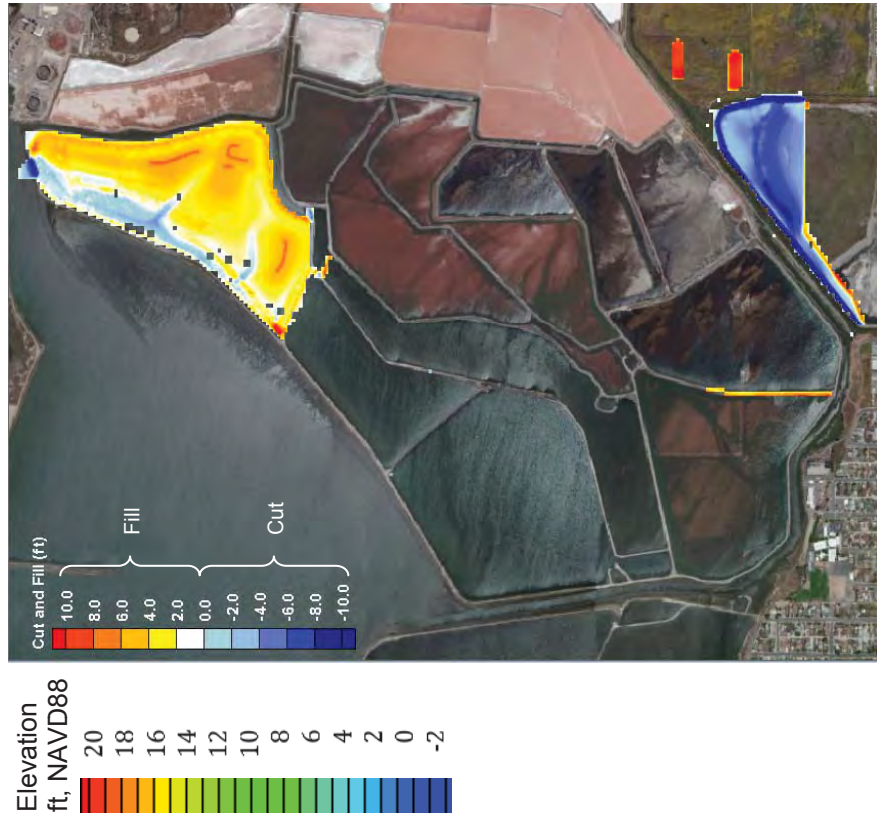
(a) Model Bathymetry

(b) Bathymetry Changes (Compared with Existing Conditions)

Figure 4.3   Model Bathymetry and Bathymetry Changes for Intertidal Alternative

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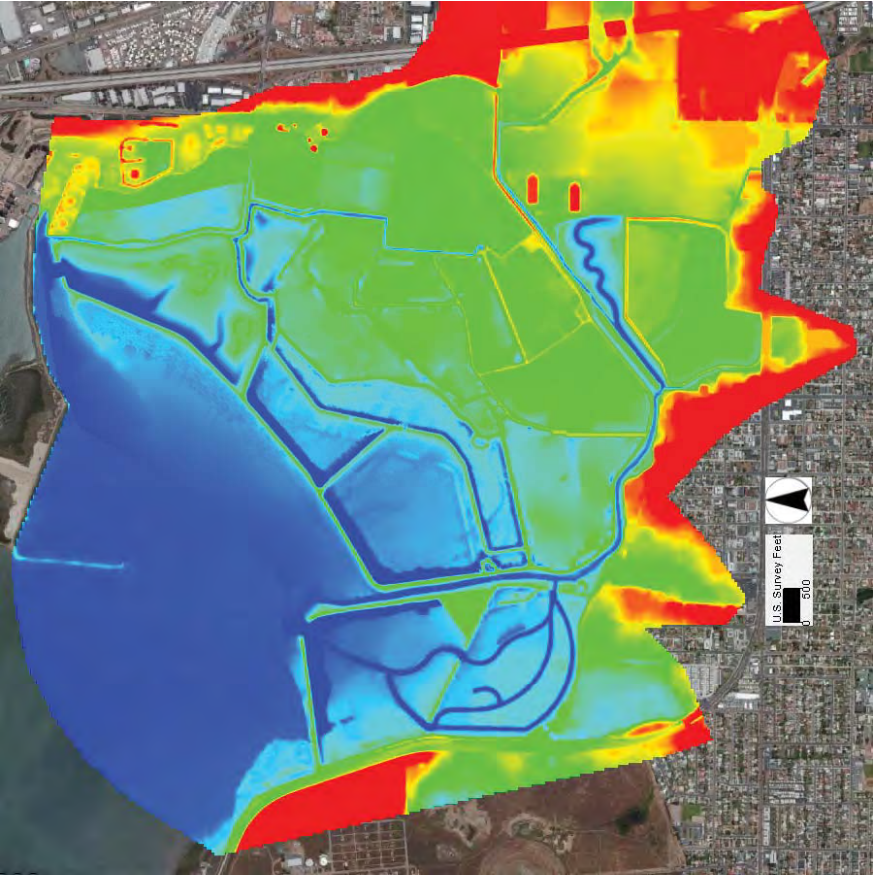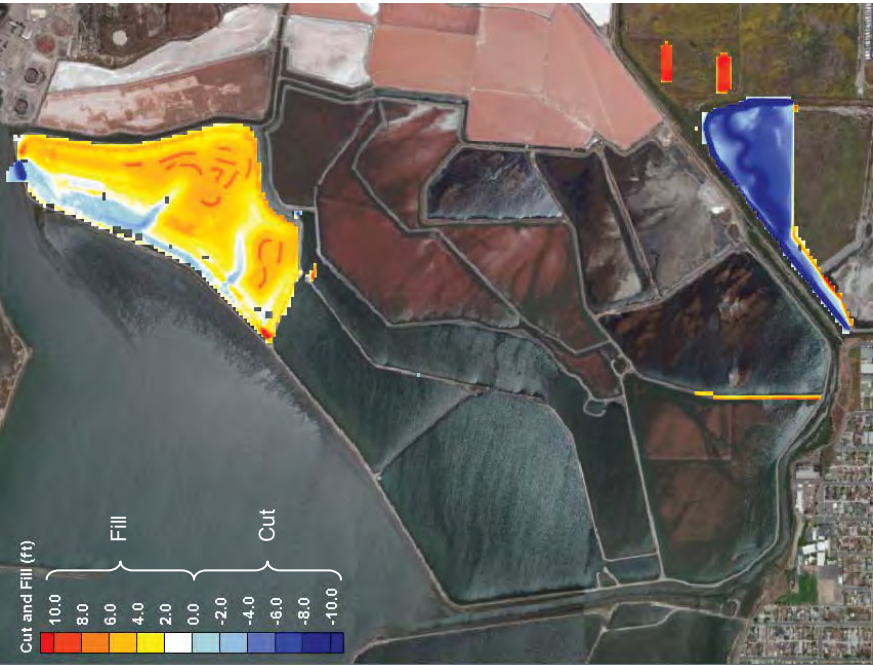
(a) Model Bathymetry

(b) Bathymetry Changes (Compared with Existing Conditions)

Figure 4.4 Model Bathymetry and Bathymetry Changes for Subtidal Alternative

4.3   MODEL BOUNDARY CONDITIONS

4.3.1   Downstream Boundary Conditions

Since the Otay River is hydraulically linked to San Diego Bay, tidal input from San Diego Bay was used in the analysis. The nearest recording water level gage operated by NOAA is located at the Navy Pier in downtown San Diego (9410170). Tidal datums for San Diego relative to Mean Lower Low Water (MLLW) and NAVD88 are listed in Table 4.2.

Table 4.2   San Diego Bay Tidal Datums for the 1983 – 2001 Tidal Epoch

DATUM	ELEVATION (FT, MLLW)	ELEVATION (FT, NAVD88)
Highest Observed Water Level (1/27/1983)	8.14	7.71
Mean Higher High Water (MHHW)	5.72	5.29
Mean High Water (MHW)	4.99	4.56
Mean Tide Level (MTL)	2.96	2.53
National Geodetic Vertical Datum (NGVD) 1929	2.51	2.08
Mean Low Water (MLW)	0.94	0.51
North American Vertical Datum (NAVD) 1988	0.43	0.00
Mean Lower Low Water (MLLW)	0.00	-0.43
Lowest Observed Water Level (12/17/1937)	-3.09	-3.52

Source: NOAA 2007

A synthetic tidal series, referred to as a parametric mean periodic (PMP) tide, was developed for the downstream boundary condition. This time series was developed by fitting a sinusoidal curve to consecutive MHHW, MLLW, MHW, and MLW water surface elevations shown in Table 4.2 over a 24-hour period, and repeating for the modeling duration. This PMP tide, as shown in Figure 4.5, is representative of the mixed diurnal, semi-diurnal tide conditions found in San Diego Bay. The highest observed tide shown in Table 4.2 was not used for flood impact analysis because the probability of having a 100-year flood event which has only a one percent chance of occurring in any given year to enter the ORF during the highest observed tide is extremely small. The use of MHHW which statistically represents a water level that is higher than about 95 percent of all the water levels in a 19-year tidal epoch is considered sufficiently conservative for flood impact analysis.

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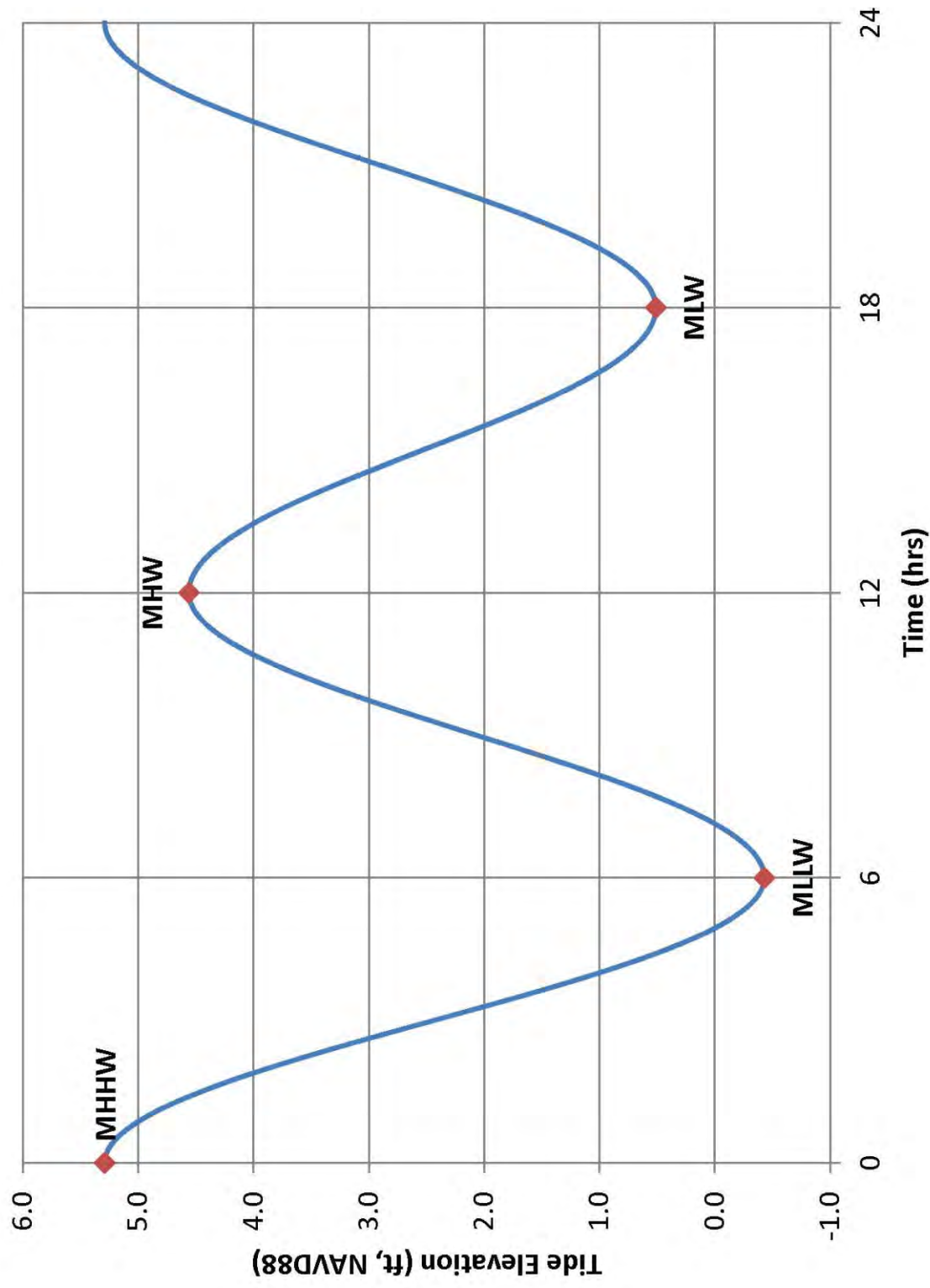
Figure 4.5   Parametric Mean Periodic Tide in San Diego Bay

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4.9

4.3.2 Upstream Boundary Conditions

Upstream boundary conditions for the Otay River, Poggi Canyon Creek, and Nestor Creek were specified as flood hydrographs with given return periods (e.g., 100-year). The return period peak discharges for the Otay River, Poggi Canyon Creek, and Nestor Creek are summarized in Table 4.3. Peak discharges of the 10-, 50-, and 100-year return periods for the Otay River and Poggi Canyon Creek were obtained from the Federal Emergency Management Agency (FEMA) peak discharge estimates (FEMA 2006). For Nestor Creek, the 10- and 50-year return period peak flows were obtained from a prior hydrodynamic modeling analysis (PWA 2003) and the 100-year return period from the FEMA estimate (FEMA 2006). For all three streams, the 15- and 25-year peak flows were interpolated based on the flows associated with the other return periods.

Simplified assumptions were made to develop the flood hydrographs. The return period peak flows were applied to a simple triangular hydrograph to produce a theoretical flood hydrograph. The peak flow was assumed to occur 12 hours after flow initiation, returning to no flow after 24 hours. This is a simplified method originally applied for the salt ponds by PWA (2003) and also adopted for an earlier study evaluating the Otay River Floodplain (Everest 2007). For example, the 100-year flood hydrographs, as shown in Figure 4.6, were developed with peak flows corresponding to the 100-year peak flows provided in Table 4.3. In the figure, the flood hydrographs are shown with two different vertical axes due to the differences in flow. The 100-year flood hydrograph for the Otay River is shown based on the left vertical axis with a peak flow of 22,000 cfs. The smaller flood hydrographs for Poggi Canyon Creek and Nestor Creek are shown with the right vertical axis with peak flows of 1,400 and 1,093 cfs, respectively. These 100-year flood hydrographs were used for the flood impact analyses described in Section 5. Other return period flood hydrographs were used for the erosion analysis discussed in Section 6.

Table 4.3   Return Period Peak Discharges

RETURN PERIOD	OTAY RIVER AT OTAY VALLEY RD	POGGI CANYON CREEK	NESTOR CREEK
10-Year	1,200[A]	220[A]	730[B]
15-Year	2,700[C]	320[C]	770[C]
25-Year	5,500[C]	520[C]	850[C]
50-Year	12,000[A]	930[A]	990[B]
100-Year	22,000[A]	1,400[A]	1,093[A]

Peak discharges in cubic feet per second (cfs)
Source: [A] FEMA 2006; [B] PWA 2003; [C] Interpolated
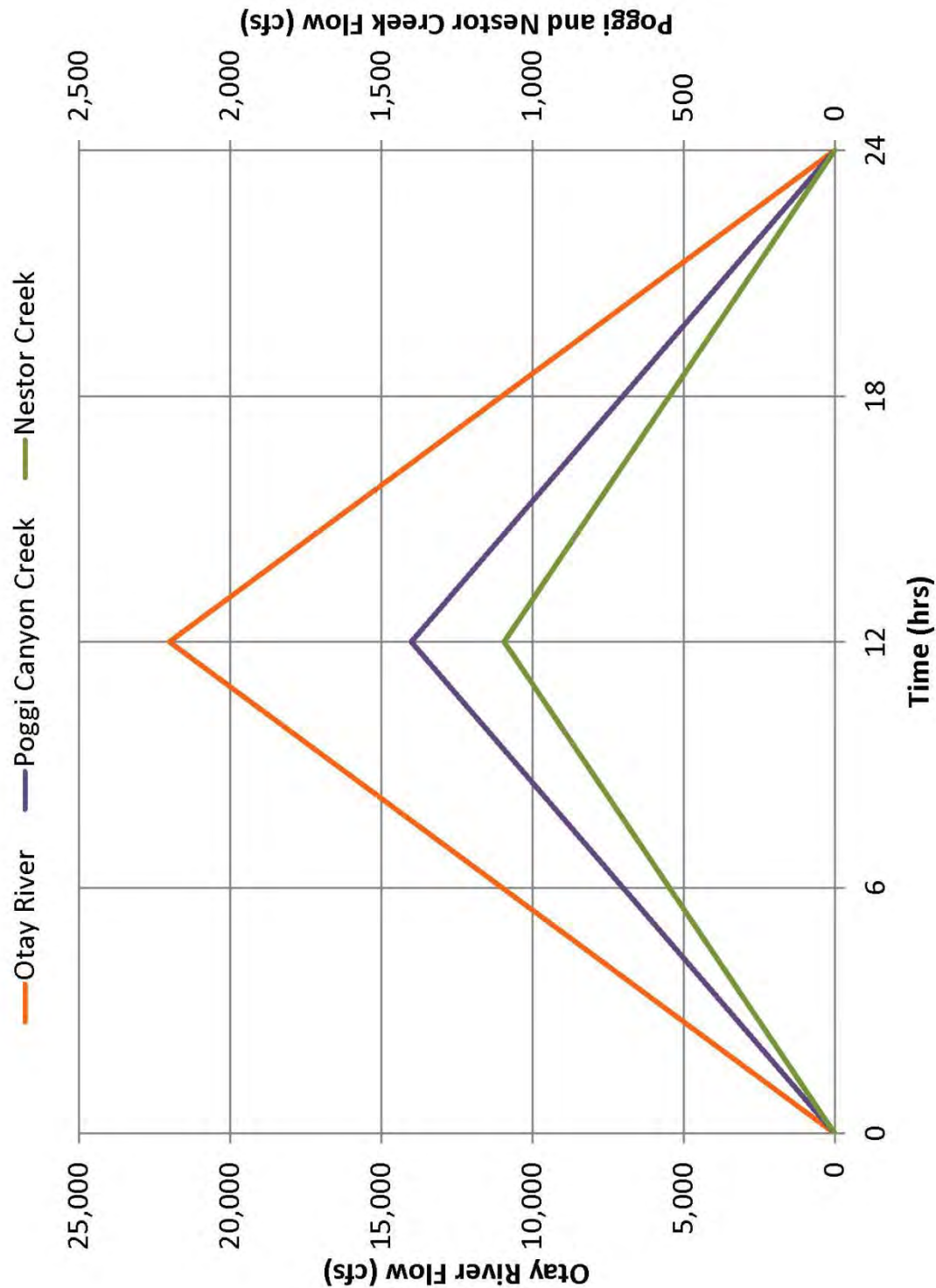
Figure 4.6   100-Year Return Period Flood Hydrographs

5. FLOOD IMPACT ANALYSES

5.1 APPROACH

The flood impact analysis was conducted to assess the impacts of the Otay River Estuary Restoration Project (ORERP) alternatives on flooding associated with the 100-year flood. Flood modeling was conducted to determine the flow pattern and water elevations during flood conditions. The results of the flood modeling under the 100-year event are summarized in Section 5.2 for existing conditions as well as the proposed project alternatives. The results under existing conditions were compared to the results under the ORERP alternatives to assess the project-induced differences as summarized in Section 5.3. An analysis of flood impacts to the Bayshore Bikeway was conducted and the results are discussed in Section 5.4. The potential impacts to erosion based on changes in flood velocities are discussed as part of the erosion analysis in Section 6.

5.2 FLOOD MODELING

The TUFLOW model was used to simulate hydrodynamic conditions of the 100-year flood for Existing Conditions as well as the Intertidal Alternative and Subtidal Alternative. Development of the flood hydrographs for Otay River, Poggi Canyon Creek, and Nestor Creek was previously discussed in Section 4.3.2. For the 100-year flood impact analyses, the flood hydrographs were timed so that the peak of the flow would coincide with MHHW to simulate high water flooding conditions, as shown in Figure 5.1. The 100-year flood hydrograph was simulated to start at hour 12 with peak occurring at hour 24 and MHHW. This timing of the flood hydrograph also allowed a spin-up period (12-hours) for the numerical model to establish hydrodynamic conditions for tidal flows. Initial water elevations were specified as 5.29 ft, NAVD88, corresponding to MHHW for tidally influenced areas including San Diego Bay and the Western Salt Ponds. An initial water elevation of 5.29 ft, NAVD88 was also specified for Ponds 12 – 15, which typically have some water. Initial water depths for these ponds ranged from 1 to 4 ft. The remaining salt ponds were assumed to be dry.
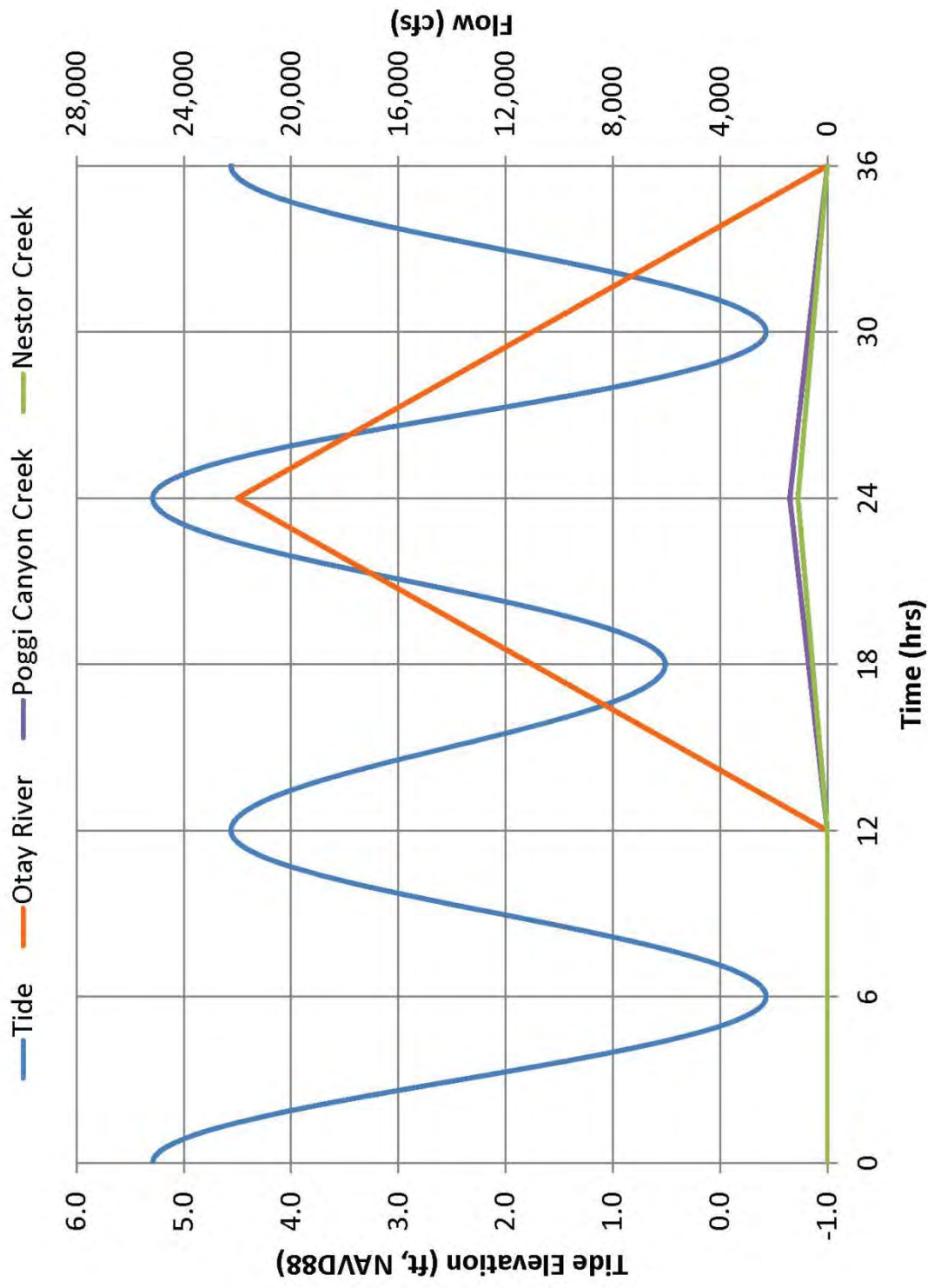
Figure 5.1 Parametric Mean Periodic Tide and 100-Year Flood Hydrographs

5.2.1 Existing Conditions

The 100-year flood was simulated to establish water elevations under Existing Conditions. Since the timing of the peak water elevation varies by location, results of the flood elevations are represented by the maximum water elevation that occurs at any point in time over the 36-hour simulation period. The spatial plot of the maximum water elevations over the entire model domain is provided in Figure 5.2. In general, the maximum water elevations follow the overall topography. Higher water elevations occur along the upper elevations along the Otay River and decrease towards the lower elevations in the ORF and salt ponds. The maximum water elevations also indicate the spatial extent of the flood inundation along the Otay River and ORF below the I-5 Bridge. The maximum water elevation was also compared with the FEMA 100-year flood map in Figure 5.3. The comparison shows that the spatial extent of the flood inundation for Existing Conditions is similar to the FEMA 100-year flood map with the exception of the Western Salt Ponds. The FEMA model represents the historical condition of Ponds 10a, 10, and 11, which were hydraulically separated from the Otay River and San Diego Bay by levees. The current model (TUFLOW) better represents the existing conditions since these three ponds were restored in 2011 resulting in hydraulic connectivity created by breaching the levees to restore tidal exchange between the ponds, San Diego Bay, and Otay River.

To illustrate the movement of the flood flow through the ORF, snapshots of the water elevations during the 100-year flood are provided in Figure 5.4. The color scale for the water elevations was selected to highlight the flood flows as indicated by the light blue to red areas. Water elevations below MHHW are shown by the blue areas, representing primarily tidal water elevations. A map of the salt ponds is provided in the lower right panel, next to the color scale. In the figure, snapshots of the water elevations in the ORF (downstream of the I-5 Bridge) are shown sequentially starting from the upper left panel, which depicts flood flows from the Otay River and Nestor Creek entering the ORF. The inset in the upper left panel indicates the timing of the five snapshots relative to the Otay River flood hydrograph. The arrival of the flood from the Otay River into the ORF occurs approximately six hours after the start of the hydrograph. This lag reflects the travel time from Otay Valley Road down to the ORF. Flows from the Otay River enter the ORF beneath the I-5 Bridge and move along the river channel towards Ponds 50 and 51. Flows from Nestor Creek move along the east edge of Pond 20A. Flood waters from Otay River and Nestor Creek continue to increase and inundate the ORF and then start to overtop levees as shown in the upper middle panel. Flows overtop the levees near the southeast corner of Pond 20A. Flood waters first enter the salt pond area through Pond 51 and start to inundate the ponds. The flood waters fill Ponds 50-54 and continue moving through the salt ponds into Ponds 41-43, 46, and 48. Farther downstream, flood flows overtop the bike path and levees at Ponds 20 and 22. At the bike path bridge, flows split westward to San Diego Bay or southward along the west side of Pond 20A. Three hours after the arrival of the flood (upper right panel), flood waters continue to inundate Ponds 20A, 20, and 22, as well as Ponds 40-48. In the lower left panel, the water elevations show the continued movement of the flood waters into the center portion of the salt ponds through Ponds 23-27. By nine hours after the arrival of

Figure 5.2   100-Year Flood Maximum Water Elevations for Existing Conditions

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Water Elevation
(ft, NAVD88)

100
90
80
70
60
50
40
30
20
10
0

U.S. Survey Feet
0   1000

Legend

100 Year Flood Zones

N
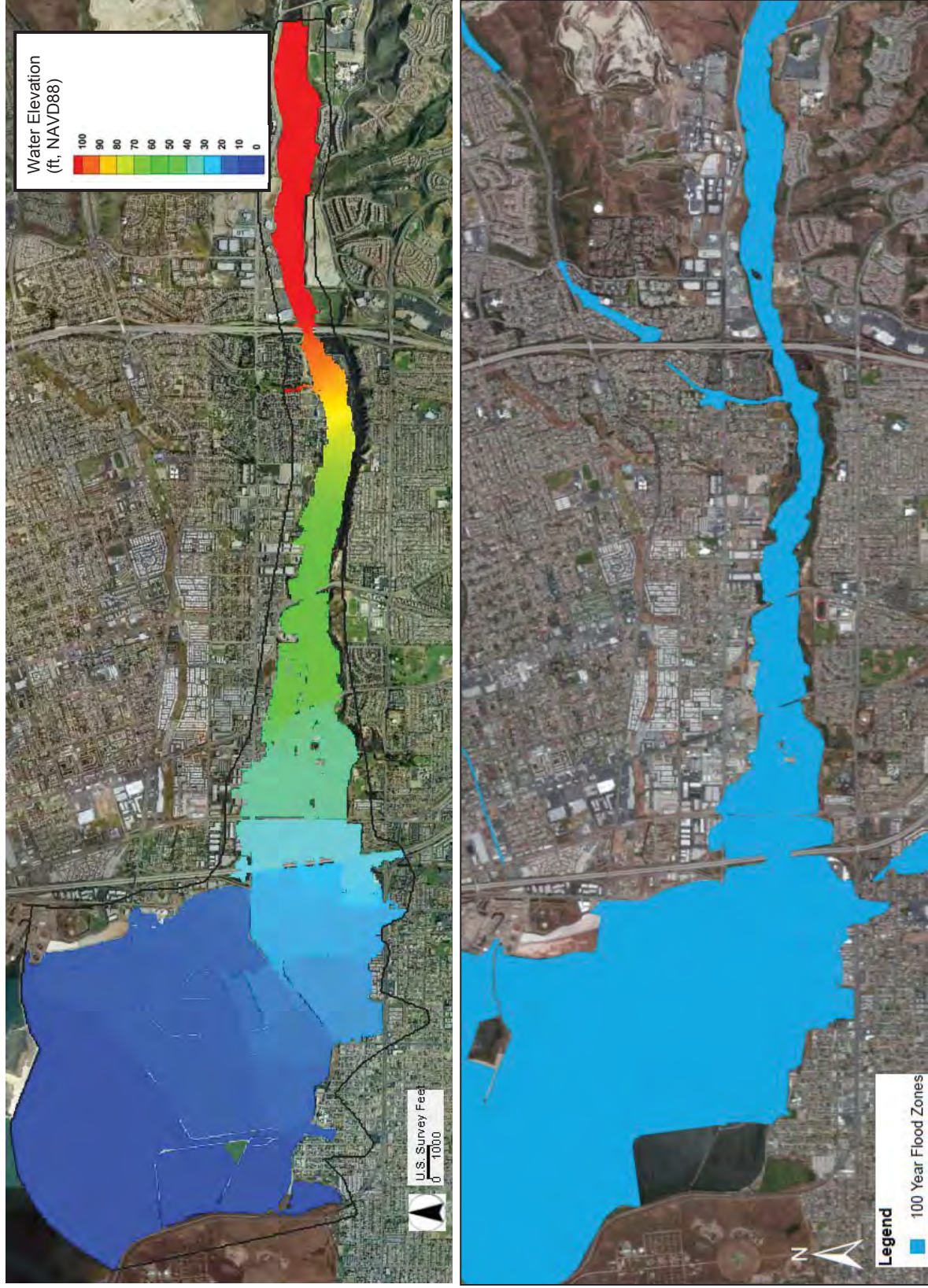
Figure 5.3   Existing Conditions Comparison with FEMA 100-Year Flood Map

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5.5

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Arrival of flood

90 min after arrival of flood

3 hrs after arrival of flood

4 hrs after arrival of flood

9 hrs after arrival of flood

Water Elevation
(ft, NAVD88)

20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
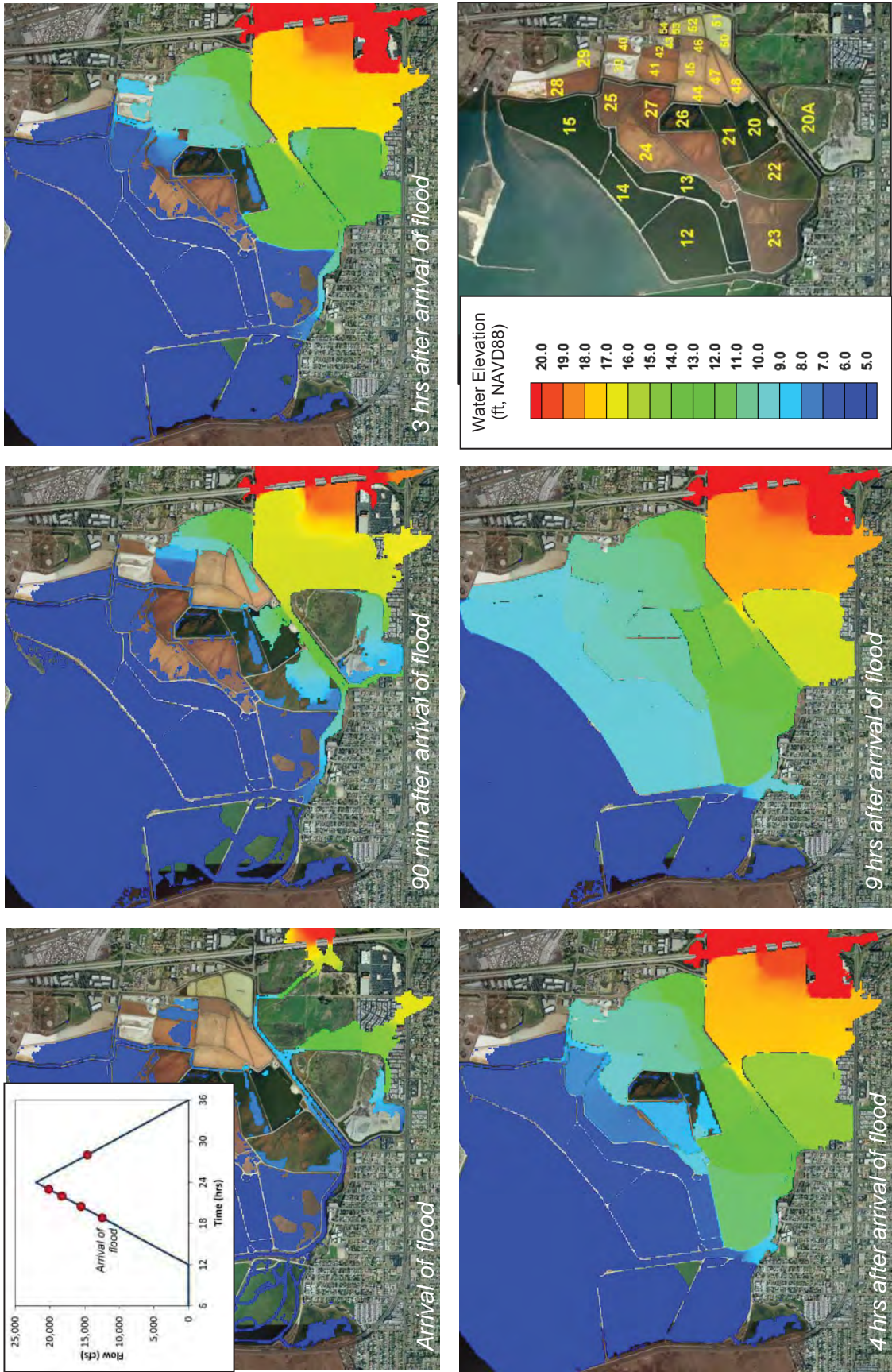
Figure 5.4   100-Year Flood Water Elevations for Existing Conditions

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the flood, flood waters inundate the remaining ponds (Ponds 12-15, 21, 26, and 28), as shown in the lower middle panel. Under Existing Conditions, the 100-year flood will inundate the ORF and salt ponds and are generally dissipated in the tidally influenced areas.

5.2.2   Intertidal Alternative

The 100-year flood was simulated for the Intertidal Alternative to establish water elevations. Maximum water elevations for the Intertidal Alternative, provided in Figure 5.5, show a general gradation from upstream to downstream. The flood inundation based on the maximum water elevations show a similar spatial extent as Existing Conditions. For the Intertidal Alternative, flood elevations upstream from the I-5 Bridge are the same as Existing Conditions. Hence, the Intertidal Alternative does not adversely impact flood conditions upstream of the I-5 Bridge.

Water elevations at different times during the 100-year flood for the Intertidal Alternative are provided in Figure 5.6. Snapshots of the water elevations are shown in the same manner as previously shown for Existing Conditions. Water elevations at the arrival of the flood from the Otay River in the ORF are shown in the upper left panel. The darker blue areas indicate tidally influenced areas including the proposed wetland area, which receives flood waters from Nestor Creek. Flood flows inundate the ORF, as depicted in the upper middle panel, then overtop the levees into the salt ponds. Flood waters enter the salt ponds through Pond 51 and subsequently fill Ponds 50-54, and 46. Flood waters also flow over the bike path and levee into Pond 22. At three hours after the arrival of the flood (upper right panel), flood waters inundate Ponds 20A and 23, while flows through Pond 51 inundate Ponds 40-43 and 48. Flows continue to inundate the salt ponds from the west side into Ponds 12-14, 24, and 27 and also from the east side into Ponds 44, 45, 47 (lower left panel). Flow waters also fill Ponds 20 and 21. By nine hours after the arrival of the flood, flood waters inundate the remaining ponds (Ponds 25, 26, 28, 29, and 30), as shown in the lower middle panel. Differences in the flow pattern of the Intertidal Alternative compared with Existing Conditions are discussed in Section 5.2.4.

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Water Elevation
(ft, NAVD88)

100
90
80
70
60
50
40
30
20
10
0

Image U.S. Geological Survey
Image Chula Vista

U.S. Survey Feet
0   1000

Figure 5.5   100-Year Flood Maximum Water Elevations for Intertidal Alternative

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Water Elevation
(ft, NAVD88)

20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0

Arrival of flood

90 min after arrival of flood

3 hrs after arrival of flood

4 hrs after arrival of flood

9 hrs after arrival of flood
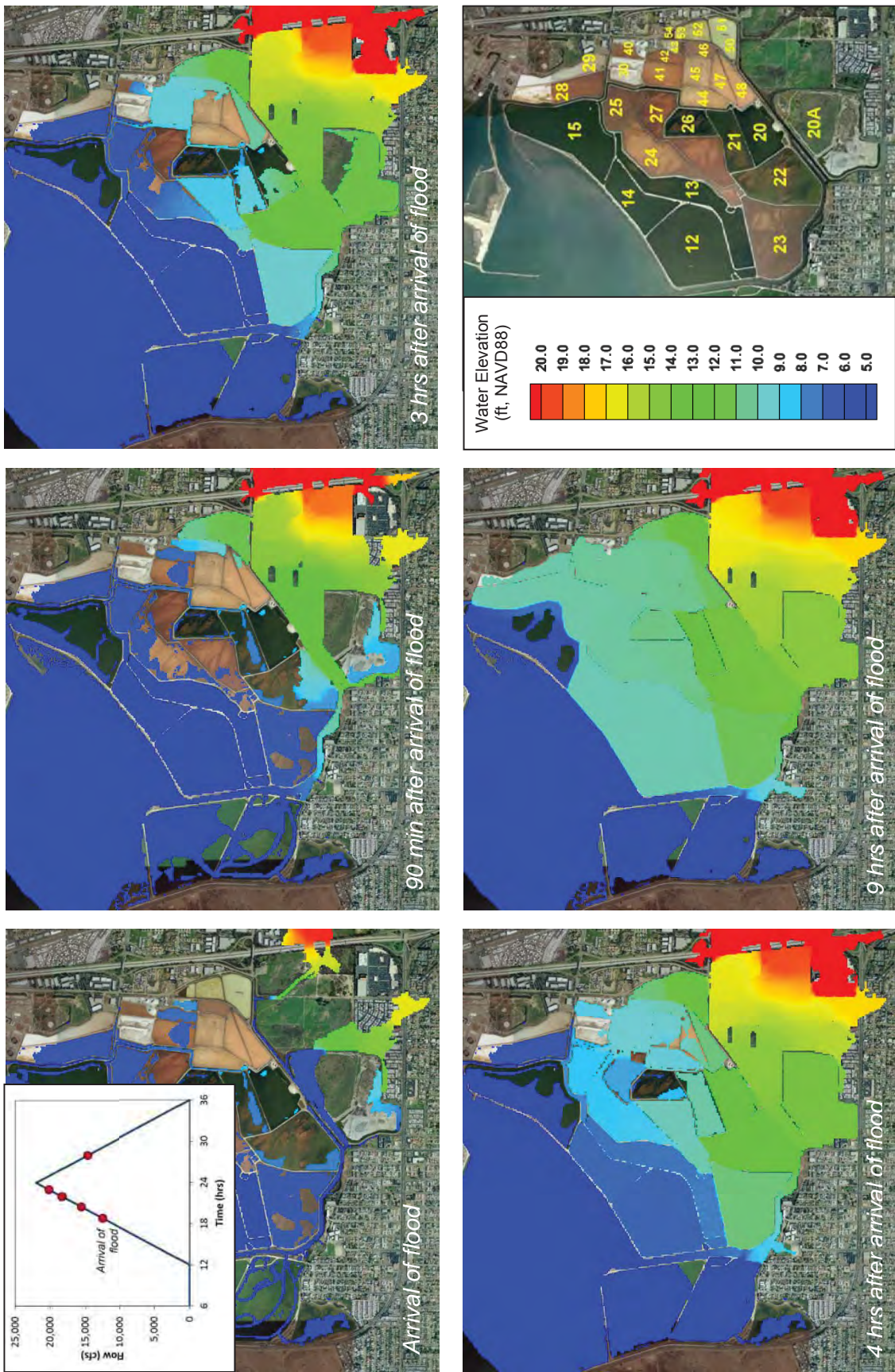
Figure 5.6   100-Year Flood Water Elevations for Intertidal Alternative

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5.2.3 Subtidal Alternative

The 100-year flood was simulated for the Subtidal Alternative to establish water elevations. The overall flood modeling results for the Subtidal Alternative are provided in Figure 5.7. The flood results for the Subtidal Alternative are similar to the Intertidal Alternative. The maximum water elevations show a general gradation from upstream to downstream with a similar spatial extent as Existing Conditions. Flood elevations along the Otay River upstream of the I-5 Bridge are the same for the Subtidal Alternative as for Existing Conditions. Hence, the Subtidal Alternative does not adversely impact flood conditions upstream of the I-5 Bridge.

Water elevations during the 100-year flood for the Subtidal Alternative are shown in Figure 5.8. The movement of the flood flows through the project area and salt ponds is similar to the Intertidal Alternative. Flood waters entering the ORF are shown in the upper left panel. Water elevations continue to increase in these areas until overtopping of the levees into Ponds 51 and 22, as seen in the upper middle panel. Three hours after the arrival of the flood, Ponds 23 and 20A becomes inundated (upper right panel). The flood waters continue through the salt ponds along the west and east sides before inundating the center ponds, as illustrated in the lower left panel. The flood eventually inundates all of the salt ponds except for Pond 15, as shown in the bottom middle panel. Differences in the flow pattern of the Subtidal Alternative compared with Existing Conditions are discussed in Section 5.2.4.

5.2.4 Comparison of Alternatives

In general, the proposed alternatives redistribute the 100-year flood flows through the salt ponds resulting in changes to the flood elevations. This redistribution of flood flows is best illustrated by comparing water elevations at two different times during the flood – approximately 90 minutes and 4 hours after arrival of the flood. In Figure 5.9, the three top panels show snapshots of water elevations approximately 90 minutes after the arrival of the flood as the flood flows move into the salt ponds for Existing Conditions, Intertidal Alternative, and Subtidal Alternative; and water elevations about four hours after the arrival of the flood flows are compared in the lower three panels. In each panel, the flood pattern is emphasized by the white arrows, which show the general direction of flow. For Existing Conditions, the flood inundates the ORF and then enters the salt ponds from Ponds 51, 20, and 22, as indicated by the three arrows. Under the proposed alternatives, flood flows would be altered by expanding the flows through the project area. As a result, flood elevations in the ORF would be reduced and flows enter the salt ponds from Ponds 51 and 22. Changes in the flow pattern through the salt ponds under the proposed alternatives are illustrated in the lower three panels. For Existing Conditions, flood waters move through the salt ponds from Ponds 51, 20, and 22 with more

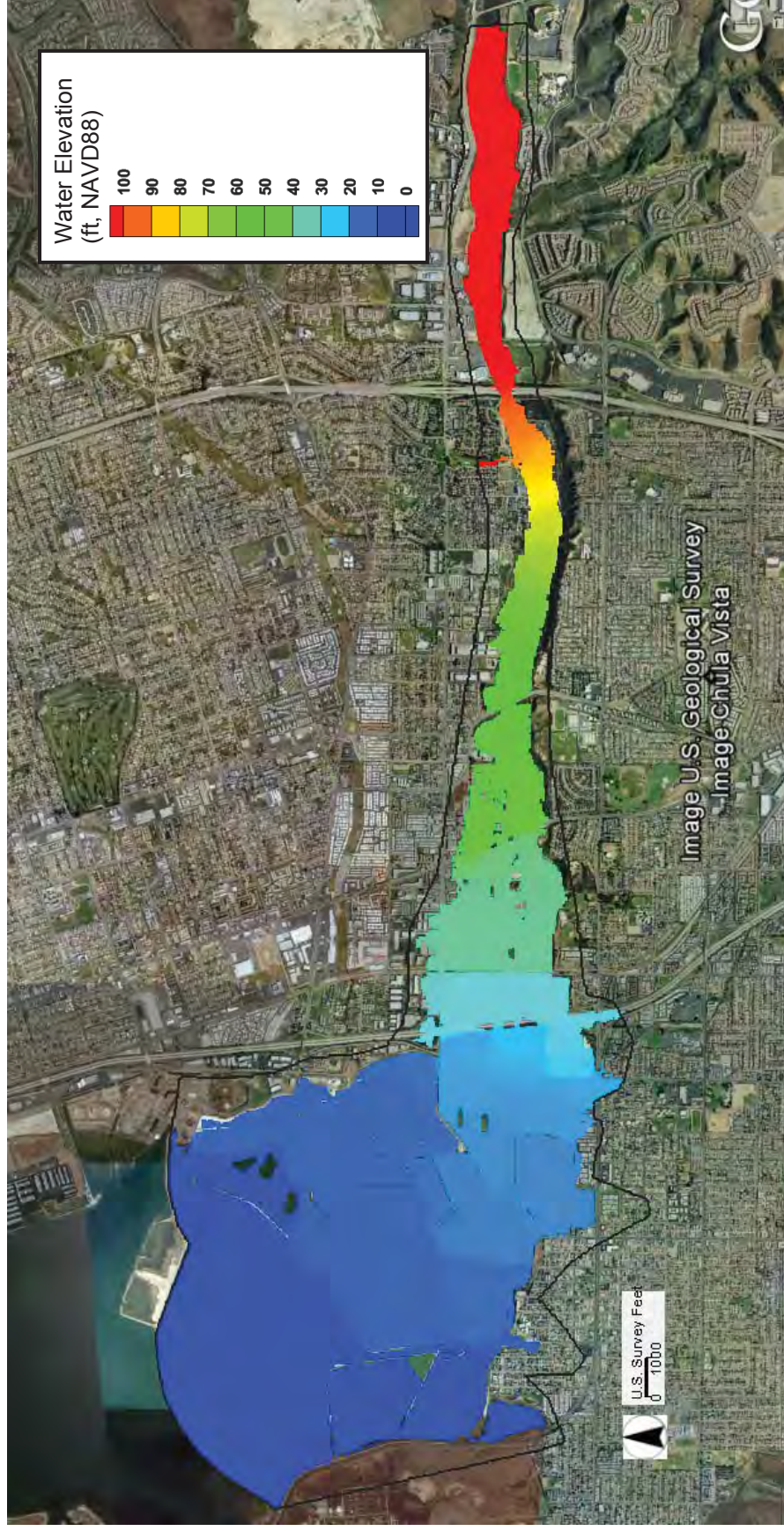**Figure 5.7   100-Year Flood Maximum Water Elevations for Subtidal Alternative**

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Water Elevation
(ft, NAVD88)

20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0

Arrival of flood

90 min after arrival of flood

3 hrs after arrival of flood

4 hrs after arrival of flood

9 hrs after arrival of flood
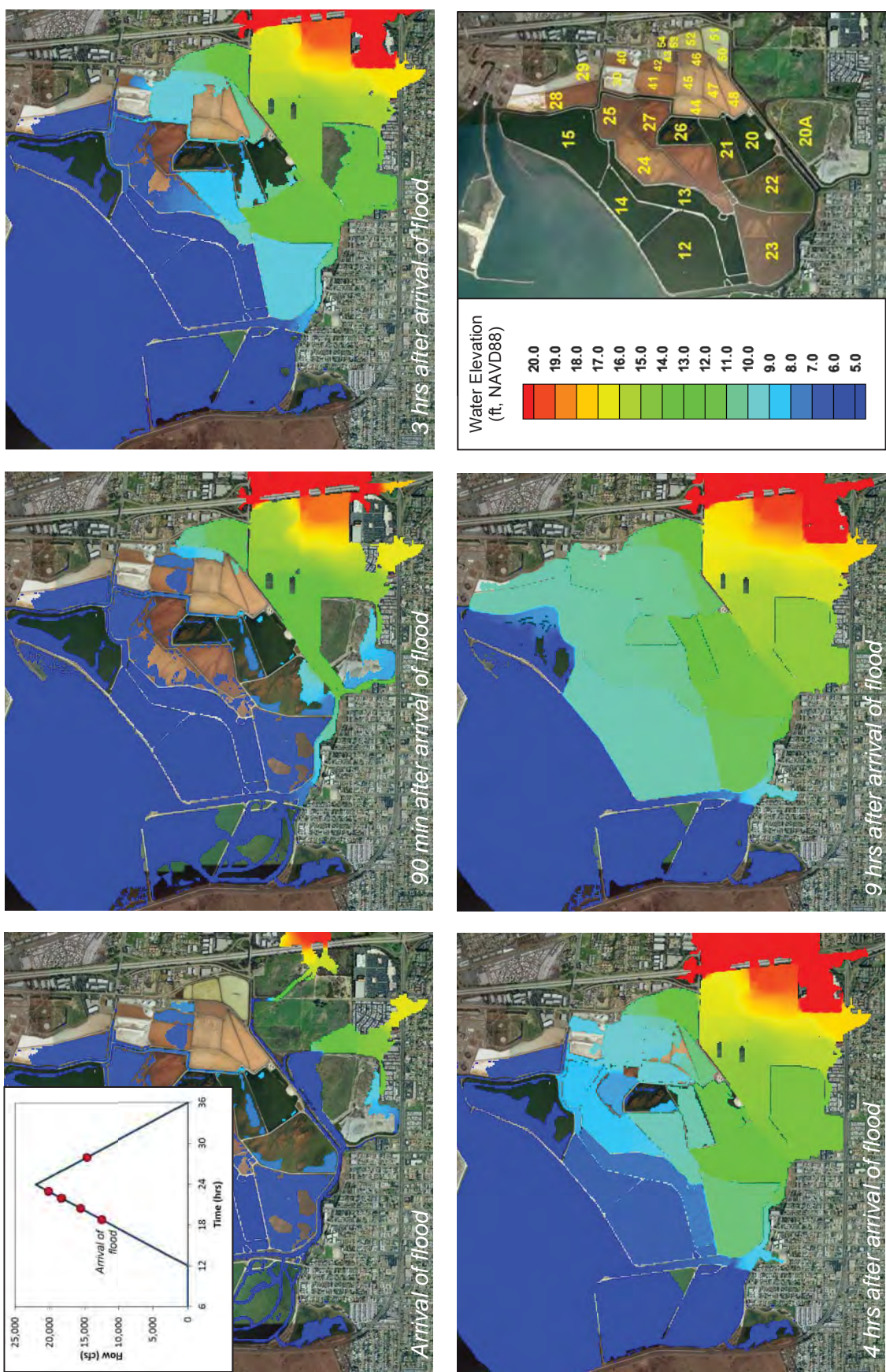
Figure 5.8   100-Year Flood Water Elevations for Subtidal Alternative

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Existing Conditions
~90 min after arrival of flood

Intertidal Alternative
~90 min after arrival of flood

Subtidal Alternative
~90 min after arrival of flood

Existing Conditions
~4 hr after arrival of flood

Intertidal Alternative
~4 hrs after arrival of flood

Subtidal Alternative
~4 hrs after arrival of flood

Figure 5.9   **Comparisons of 100-Year Flood Water Elevations**

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flows from Pond 51. Under the proposed alternatives, a greater amount of flooding occurs from the west side of the salt ponds entering from Pond 22. This is highlighted by the two white arrows on the west side of the salt ponds. This results in higher water elevations in Ponds 12 – 14. Lower flows through the center of the salt ponds (Pond 20) are indicated by the lower water elevation compared to Existing Conditions. On the east side of the salt ponds (Pond 51), Existing Conditions show a larger inundated area compared to the alternatives also indicating lower flows. The increase in flood elevations in Ponds 12 – 14 is discussed further below.

Flood elevations were evaluated based on the maximum water elevations that occurred over the 100-year flood event. The maximum water elevations downstream of the I-5 Bridge for Existing Conditions, Intertidal Alternative, and Subtidal Alternative are compared in Figures 5.10. These results are the same as previously shown for the entire model domain in Figures 5.2, 5.5, and 5.7, but the color scale has been changed to highlight the differences in water elevations for the area downstream of the I-5 Bridge. Comparisons between Existing Conditions and the proposed alternatives show differences in the spatial extent of flooding and flood elevations. The flooding of the residential area along Palm Avenue (south of Pond 20A) under Existing Conditions is eliminated under either alternative. Additional flooding would occur for both alternatives at Pond 29, which is not flooded under Existing Conditions. Differences in flood elevations from Existing Conditions are apparent in the ORF and project areas (area south of the bike path). Both alternatives would result in lower water elevations in the ORF and project areas compared to Existing Conditions. Lower water elevations were also found in Pond 15, which is isolated from the flood waters under the alternatives. Higher water elevations for the alternatives are shown in Ponds 12 – 14 and 28. Under Existing Conditions, flood waters overtop the levees into San Diego Bay along Ponds 12, 14, and 15. Under the alternatives, overtopping of the levees into San Diego Bay occurs only along Ponds 12 and 14. Overtopping of the Ponds 12 and 13 levees adjacent to the Otay River occur under the alternatives. For Pond 23, flow over the levees into the river was determined for Existing Conditions and the alternatives. These changes in flood elevations are attributed to the redistribution of flows through salt ponds.

5.3   FLOOD IMPACTS

Flood impacts of the proposed alternatives focused on the differences in the maximum flood elevations from Existing Conditions. The differences, as shown in Figure 5.11, were calculated as the maximum flood elevation for the alternative less the maximum flood elevation for Existing Conditions. In the figure, the white areas indicate no change in maximum water elevation from Existing Conditions. Positive values indicate higher flood elevations for the alternative compared to Existing Conditions, while negative values indicate lower flood elevations. Yellow areas indicate higher flood elevations under the proposed alternatives compared to Existing Conditions. The highest increases in flood elevations are found in Ponds 12 – 14 and 28. Increases in flood elevations are also determined for the bike path along Pond 22. The yellow

**Water Elevation
(ft, NAVD88)**

5   7   9   11   13   15   17   19

Existing Conditions

Intertidal Alternative

Subtidal Alternative
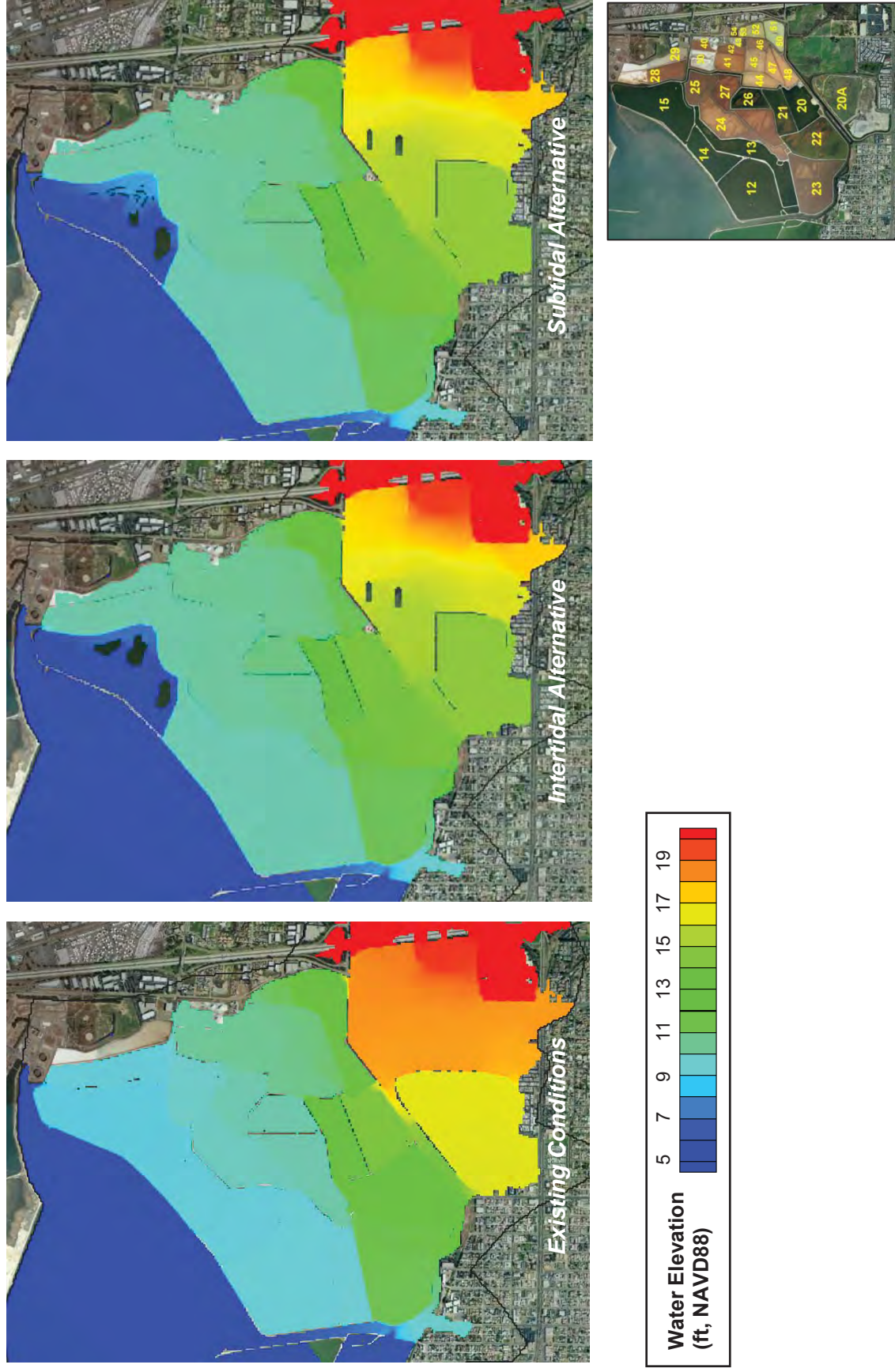
Figure 5.10   Comparisons of 100-Year Flood Maximum Water Elevations

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Subtidal Alternative

Intertidal Alternative

Water
Level
Difference
(ft)

-4 -3 -2 -1   0   1   2   3   4

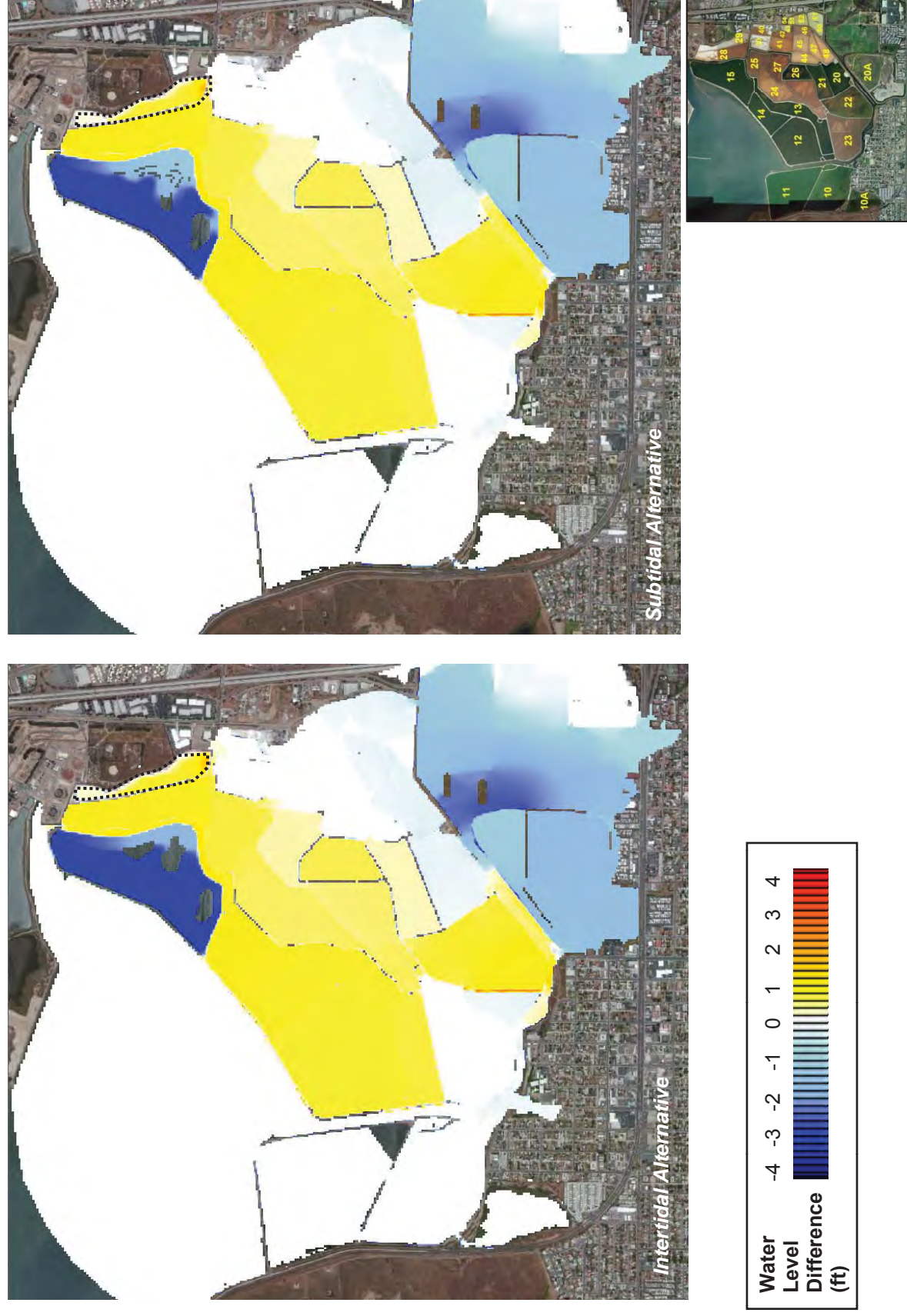**Figure 5.11   100-Year Flood Impacts – Change in Maximum Water Elevations when Compared with Existing Conditions**

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area in Pond 29 (outlined in dotted lines) is an area flooding occurs under the alternatives but not under Existing Conditions. The water elevation difference for that area is calculated as the difference in water elevation between the alternatives and the ground elevation of the existing condition. In the figure, lighter blue areas indicate reductions in flood elevations, which primarily occur in the floodplain and project areas. Reductions in flood elevations are also observed within Pond 15 as well as the levee between Pond 15 and San Diego Bay. The darker blue indicates areas that are flooded under Existing Conditions, but are no longer flooded under the alternative such as the stockpiles, Pond 15, and the residential area near Palm Avenue.

Comparisons between the Intertidal and Subtidal Alternatives show similar flood impacts in the salt ponds for both alternatives. The Subtidal Alternative results in higher flood elevations for the south end of the bike path along Pond 22.

5.3.1 Salt Ponds 14 and 28

As mentioned previously, the proposed alternatives will result in higher flood elevations compared to Existing Conditions in Ponds 12 to 14, 28, and 29 due to changes in the flow distribution through the salt ponds. The higher levees around Pond 15 under the proposed alternatives also contribute to the higher water elevations by reducing the flood area. The higher water elevations result in higher flows over the Pond 12 and 14 levees into San Diego Bay as well as higher flows into Ponds 28 and 29. Examples of the increase in flood elevations at Ponds 14 and 28 are provided in Figure 5.12. Time series of water elevations during the 100-year flood are shown for Existing Conditions (blue line), Intertidal Alternative (orange line), and Subtidal Alternative (green line). The black-dashed line indicates the average levee elevation so water elevations above this line indicate overtopping of the levee. Water elevations in Pond 14 are compared in the top panel. The elevation of the levee between Pond 14 and San Diego Bay ranges from 7.5 to 9.5 ft, NAVD88, with an average of about 8.5 ft, NAVD88. A portion of the levee is overtopped under Existing Conditions, while essentially the entire length of the levee would be overtopped under the proposed alternatives. For Pond 28 shown in the lower panel, water elevations for Existing Conditions is lower than the levee elevation between Pond 28 and Pond 29, but under the proposed alternatives, flood elevations at Pond 28 would be higher than the levee between Pond 28 and Pond 29. Hence, as discussed earlier, Pond 29 is not flooded under existing alternatives but would be flooded with the proposed alternatives.
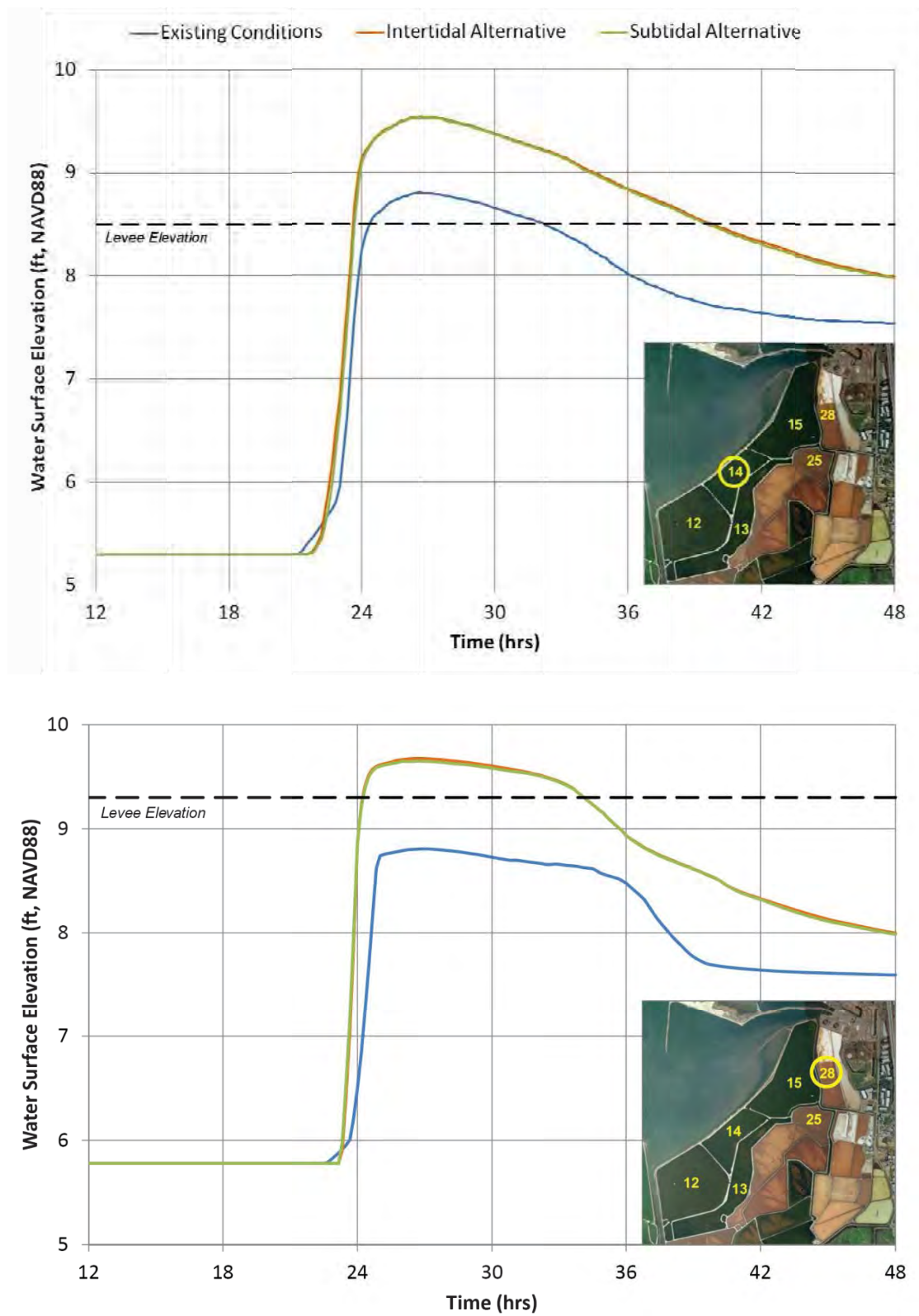
Figure 5.12   100-Year Flood Elevations at Ponds 14 and 28

5.4   BIKE PATH IMPACTS

The 100-year flood modeling described previously showed changes along the bike path in the maximum 100-year flood elevation due to the redistribution of flows under the proposed alternatives. A closer look of the changes to the maximum flood elevations along the south end of the bike path is shown in Figure 5.13. The inset photo shows the extent of the changes for each alternative. In the figure, the yellow area indicates higher flood elevations compared to Existing Conditions, while blue areas indicate lower flood elevations. In general, the 100-year flood elevations would decrease at the center portion of the bike path along Pond 20, but increase at the southern end of the bike path along Pond 22. The higher flood elevations are due to the redistribution of the flood flows. Flood flows that would overtop into Pond 20 under Existing Conditions would be diverted downstream into the wetland area and Pond 22. Although the proposed alternatives would increase flood elevations for the 100-year flood, flood impacts would be reduced for smaller floods events.

Additional flood simulations were conducted to further evaluate potential flooding impact of the bike path due to the proposed alternatives. Flood impacts were evaluated in terms of when flood water levels would exceed the bike path elevation in addition to changes in flood water levels from existing condition during the 100-year flood event. Additional flood modeling was conducted for the 10-, 15-, 25-, and 50-year return period floods to determine the minimum flood size that would result in flooding of the bike path. These additional floods were simulated in the same manner as the 100-year flood with flood hydrographs developed for each return period as discussed previously in Section 4.3.2. It was determined that depending on the location, flooding along the bike path would begin between the 10-year and 15-year flood under existing condition. However, as illustrated in Figure 5.14, with the proposed alternatives, flooding of the bike path would not occur up to the 15-year flood event. In Figure 5.14, water elevations for the 15-year and 100-year floods at three locations along the bike path under existing and with project alternative conditions are compared. The three locations for the comparison of water elevations are shown in the inset at the bottom of the figure. Time series of water elevations for the 15-year flood are shown in the upper three panels and the corresponding water elevations for the 100-year flood are contained in the lower three panels. In the figure, time series for Existing Conditions, Intertidal Alternative, and Subtidal Alternative are indicated by the blue, orange, and green lines, respectively. The black-dashed line is the ground elevation of the bike path at each location. As shown in the figure, at Location 1, flood elevations under the proposed alternatives would be higher than Existing Conditions for both the 15- and 100-year floods. At Location 2, flood elevations would be reduced under proposed conditions for the 15-year flood, whereas, flood elevations would be higher than Existing Conditions for the 100-year flood. At Location 3, both the 15- and 100-year flood elevations for the proposed alternatives would be less than Existing Conditions.

Intertidal Alternative

Subtidal Alternative

Water
Elevation
Change
(ft)

| -2.0 | -1.6 | -1.2 | -0.8 | -0.4 | 0.0 | 0.4 | 0.8 | 1.2 | 1.6 | 2.0 |
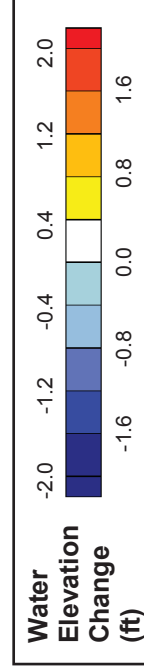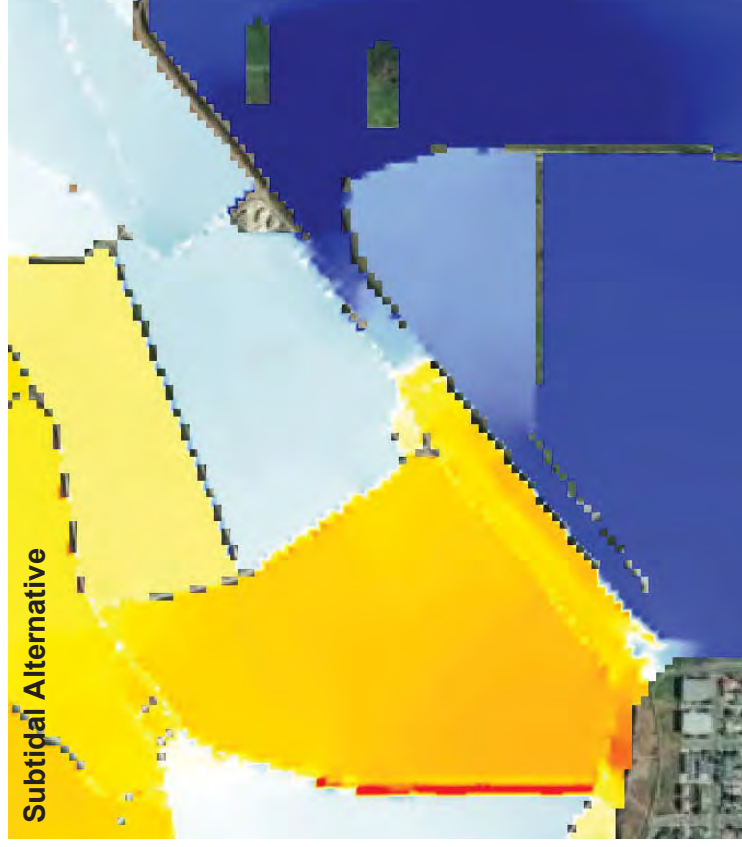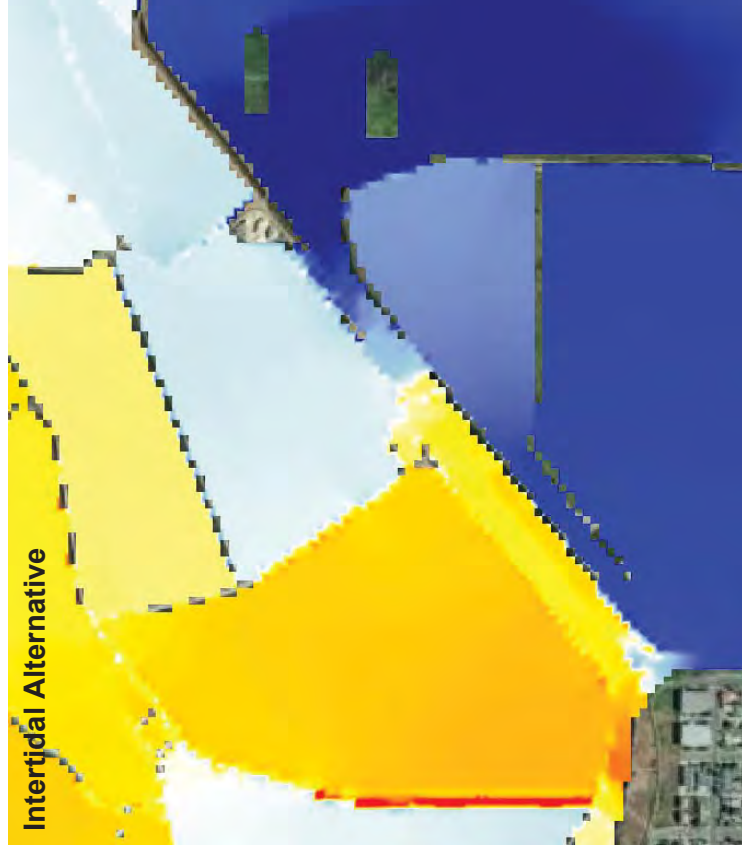
Figure 5.13   100-Year Flood Impacts along Bike Path

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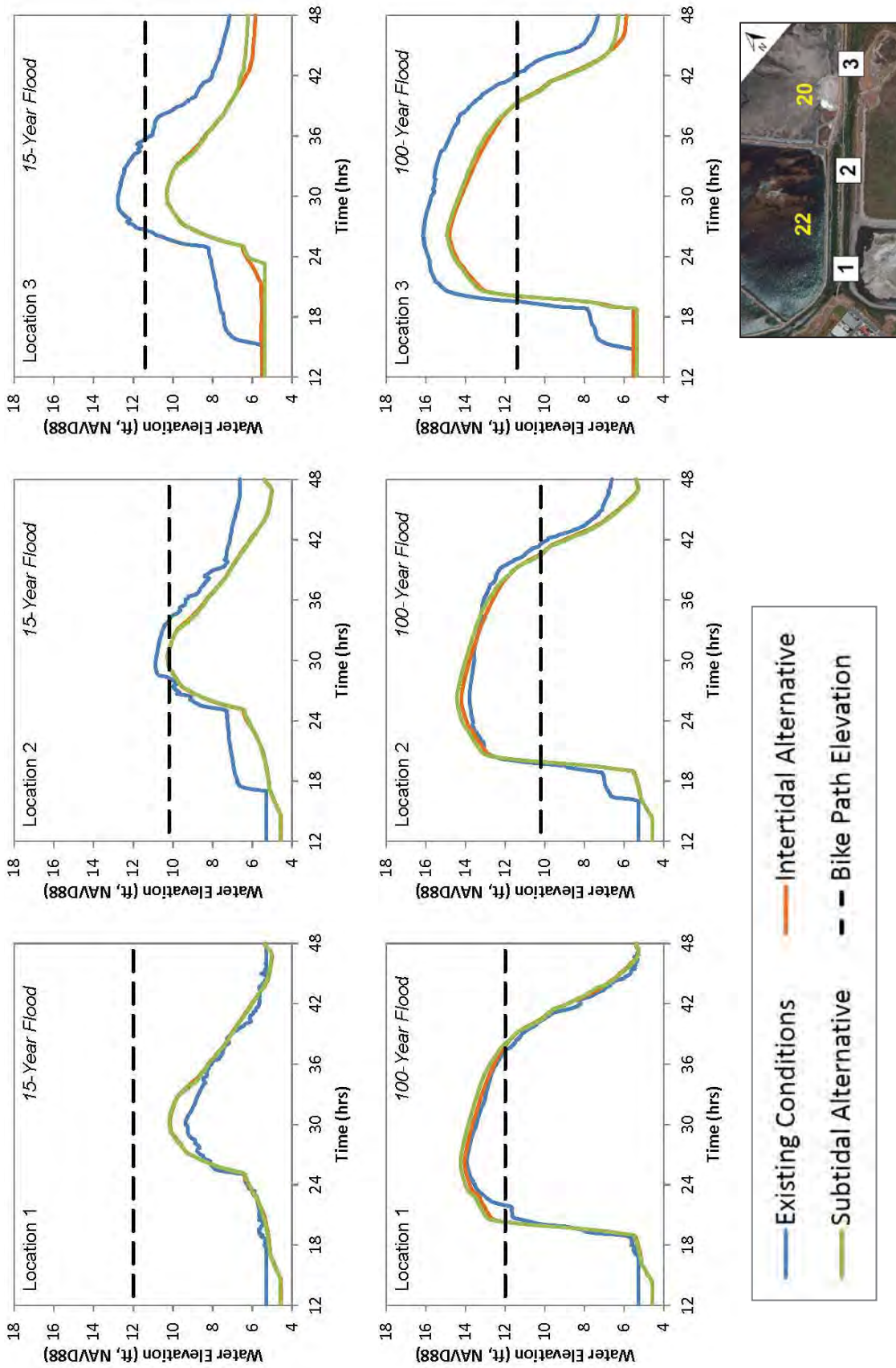
Figure 5.14   15-Year and 100-Year Flood Elevations along Bike Path

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5.21

Flooding of the bike path occurs when water levels exceed the bike path elevation and overtops the bike path. In Figure 5.14, water elevations above the black-dashed line indicate flooding of the bike path. The bike path would be flooded by a 100-year event under both existing and proposed conditions. However, the proposed alternatives would reduce flooding of the bike path for a 15-year flood event. No flooding occurs for the 15-year flood event at Location 1 under both existing and proposed conditions, but at the other two locations, the proposed alternatives would alleviate flooding of the bike path for the15-year flood event. In summary, the proposed alternatives would not alleviate flooding of the bike path for extreme flood events (e.g., 100-year flood), but would prevent flooding of the bike path for smaller and more frequent flood events (e.g., 15-year flood).

5.5   SUMMARY

Flood modeling was conducted to establish the flow pattern and water elevations during flood events. The flood impact analysis was conducted for the 100-year flood from the Otay River, Poggi Canyon Creek, and Nestor Creek. Flood conditions were simulated for Existing Conditions, Intertidal Alternative, and Subtidal Alternative and then compared to evaluate changes in flow pattern and maximum water elevations. For Existing Conditions, the flood inundates the ORF and then enters the salt ponds from Ponds 51, 20, and 22. The salt ponds are filled from primarily the west and east sides before overtopping the levees into San Diego Bay. Under the alternatives, flood flows are redistributed through the project area and enter the salt ponds through Ponds 51 and 22. A greater amount of flooding occurs from the west side of the salt ponds compared to the east side inundating all the ponds except for Pond 15, which is isolated from flood flows. Higher flood elevations in the northern portion of the salt ponds results in greater flows overtopping into San Diego Bay along Ponds 12 and 14 as well as greater flows into Ponds 28 and 29.

Reductions in flood impacts were determined for the ORF and project areas, Pond 20A, Pond 20, and Pond 15. Along the bike path, the proposed alternatives would reduce flood elevations at the north end of the bike path adjacent to Pond 48. In general, the proposed alternatives would not change flood elevations in tidally influence areas, including the Western Salt Pond Restoration area (formerly Ponds 10A, 10, and 11). Flood impacts of the proposed alternatives were determined for Ponds 12, 13, 14, 28, and 29. Increases in 100-year flood elevations were also found for the south end of the bike path along Pond 22.

Additional flood simulations for flood events with different return periods were conducted to assess flood impacts in terms of flooding of the bike path. The proposed alternatives would not alleviate flooding of the bike path for extreme flood events (e.g., 100-year flood), but would prevent flooding of the bike path for smaller flood events (e.g., 15-year flood).

6. EROSION IMPACT ANALYSIS

6.1 APPROACH

The erosion impact analysis was conducted to identify erosion (scour) associated with the proposed alternatives. In addition to water levels, the flood model TUFLOW provided velocities during flood conditions. The erosion of sediment is dependent primarily on the water velocity and sediment grain size. In general, higher velocities will correspond with greater erosion. Erosion impacts were qualitatively assessed based on change in velocities under the proposed conditions compared to Existing Conditions. Areas with lower velocities than Existing Conditions are expected to have reduced erosion, while areas with higher velocities are expected to have greater erosion. The velocity results for the 100-year flood flow are presented in Section 6.2. Erosion impacts are discussed in Section 6.3, including impacts along the bike path.

6.2 VELOCITY RESULTS

6.2.1 Existing Conditions

The 100-year flood modeling was used to establish flood velocities under Existing Conditions. Results of the flood velocities are represented by the maximum velocity that occurs at any point in time over the 36-hour simulation period. The spatial plot of the maximum velocities over the entire model domain is provided in Figure 6.1. In general, the highest velocities occur along the Otay River channel and levees. Higher velocities, as indicated by the red color, are shown along the entire stretch of the Otay River from the I-5 Bridge to San Diego Bay. These velocities range from about 7 to 10 ft/sec. Similarly, higher velocities are observed along the salt pond levees attributed to the flood flows overtopping the levees. Higher velocities also occur along the levee separating Ponds 14 and 15 with San Diego Bay due to overtopping of the levees.

6.2.2 Intertidal Alternative

The maximum velocities over the entire model domain for the Intertidal Alternative are provided in Figure 6.2, which shows the highest velocities occurring along the river channel and levees. In the upper portion of the Otay River, maximum velocities for the Intertidal Alternative are similar to Existing Conditions indicating the alternative would not cause any erosion impacts upstream of the I-5 Bridge.

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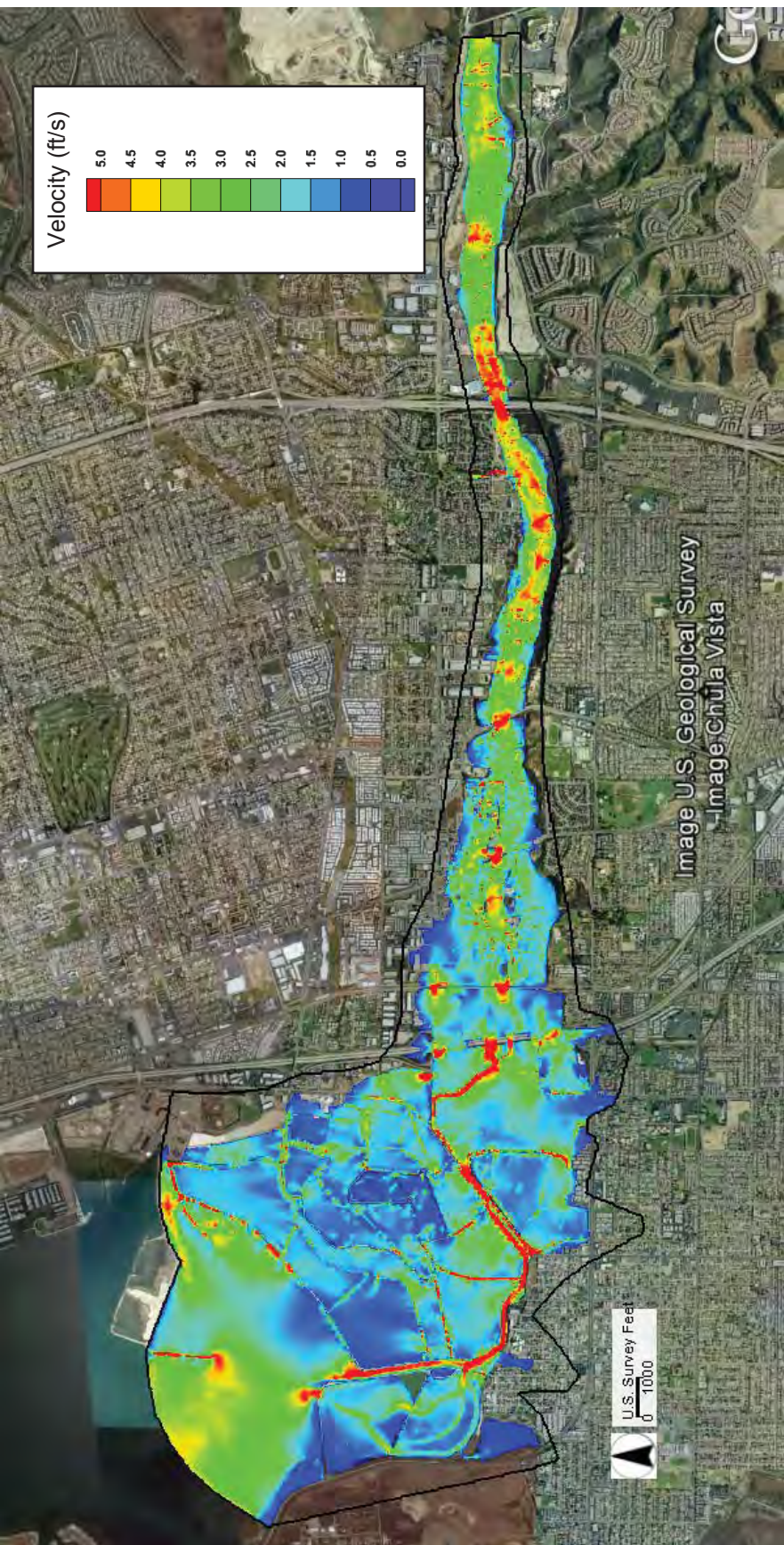
Figure 6.1   100-Year Flood Maximum Velocities for Existing Conditions

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6.2

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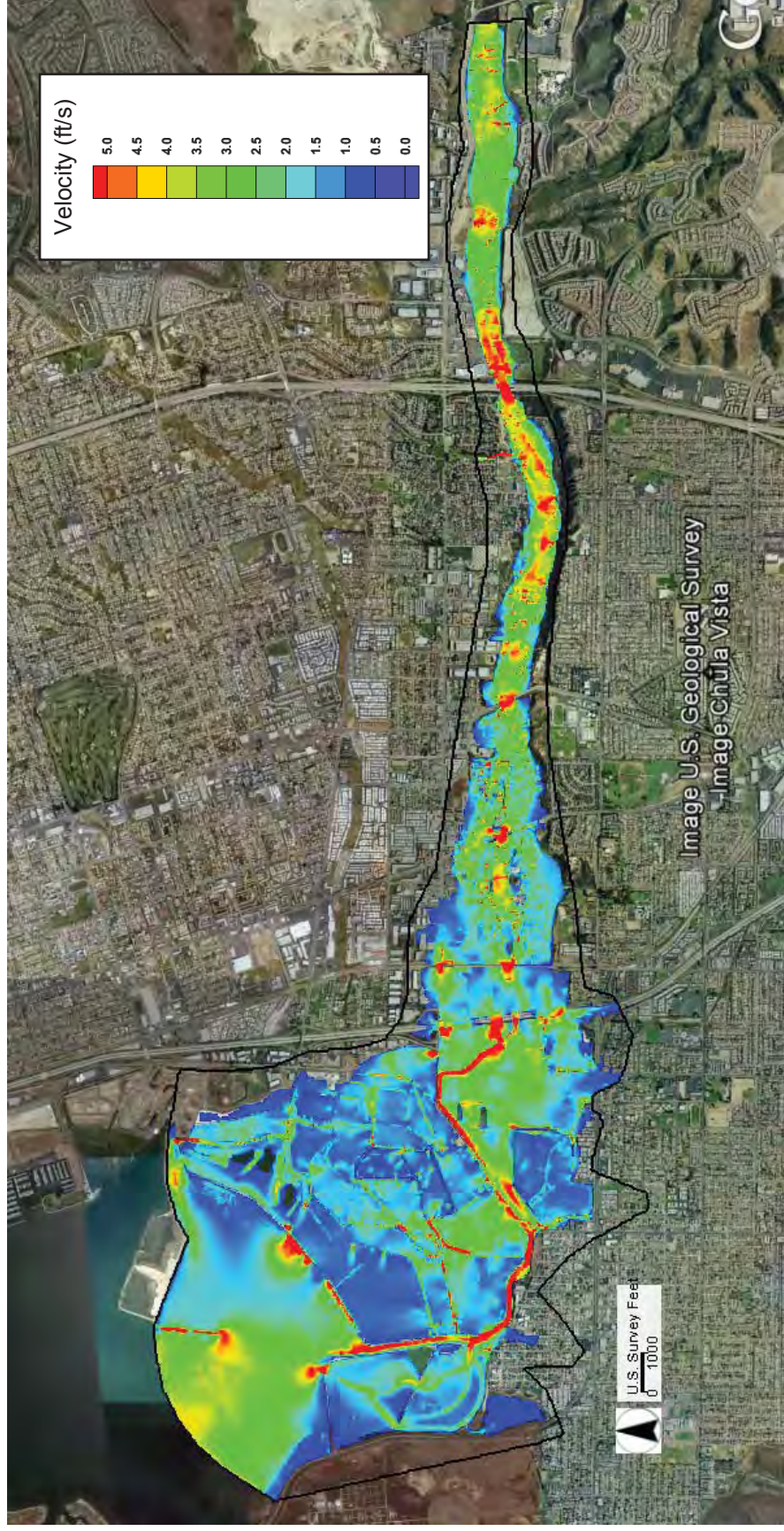
Figure 6.2   100-Year Flood Maximum Velocities for Intertidal Alternative

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A comparison of maximum flood velocities downstream of the I-5 Bridge for Existing Conditions and Intertidal Alternative is provided in Figure 6.3. In the figure, the color scale has been selected to differentiate the high and low velocities. Flood velocities under the Intertidal Alternative are similar in magnitude to Existing Conditions, but locations of higher and lower velocities differ. As expected, differences in the flood velocities are shown throughout the project area due to the changes in grading. Differences in flood velocities also occur along the bike path adjacent to Ponds 48, 20, and 22, coinciding with differences in flood elevations. High velocities are also observed between the stock pile areas. Along the pond levees separating the salt ponds and San Diego, differences in flood velocities are apparent. Under Existing Conditions, higher velocities occur along Ponds 14 and 15 due to overtopping of the levees. Under the Intertidal Alternative, lower velocities occur along Pond 15 since with the new tidal inlet, no overtopping occurs from Pond 15. However, higher velocities are shown along Ponds 12 and 14 under the Intertidal Alternative due to additional overtopping.

6.2.3   Subtidal Alternative

For the Subtidal Alternative, the maximum velocities during the 100-Year flood are shown in Figure 6.4. Similar to the Intertidal Alternative, the highest velocities occur along the Otay River channel and levees. Above the I-5 Bridge, maximum velocities are similar to Existing Conditions, indicating no erosion impacts.

A comparison of the velocities for the Subtidal Alternative and Existing Conditions in the ORF downstream of the I-5 Bridge is shown in Figure 6.5. Overall, flood velocities under the Subtidal Alternative are similar in magnitude to Existing Conditions, but locations of higher and lower velocities vary. Similar to the Intertidal Alternative, differences in flood velocities are apparent throughout the ORF and project areas due to changes in the grading. High velocities are also observed between the stock pile areas. Along the bike path, higher velocities occur at the south end of the bike path under Existing Conditions, but higher velocities occur at the north end under the Subtidal Alternative. Differences in flood velocities are also noticeable along the levees adjacent to San Diego Bay. Higher velocities mainly occur along Pond 15 under Existing Conditions, but occur along Ponds 12 and 14 under the Subtidal Alternative.

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Intertidal Alternative

Existing Conditions

Velocity (ft/s)

0 1 2 3 4 5 6 7 8 9 10
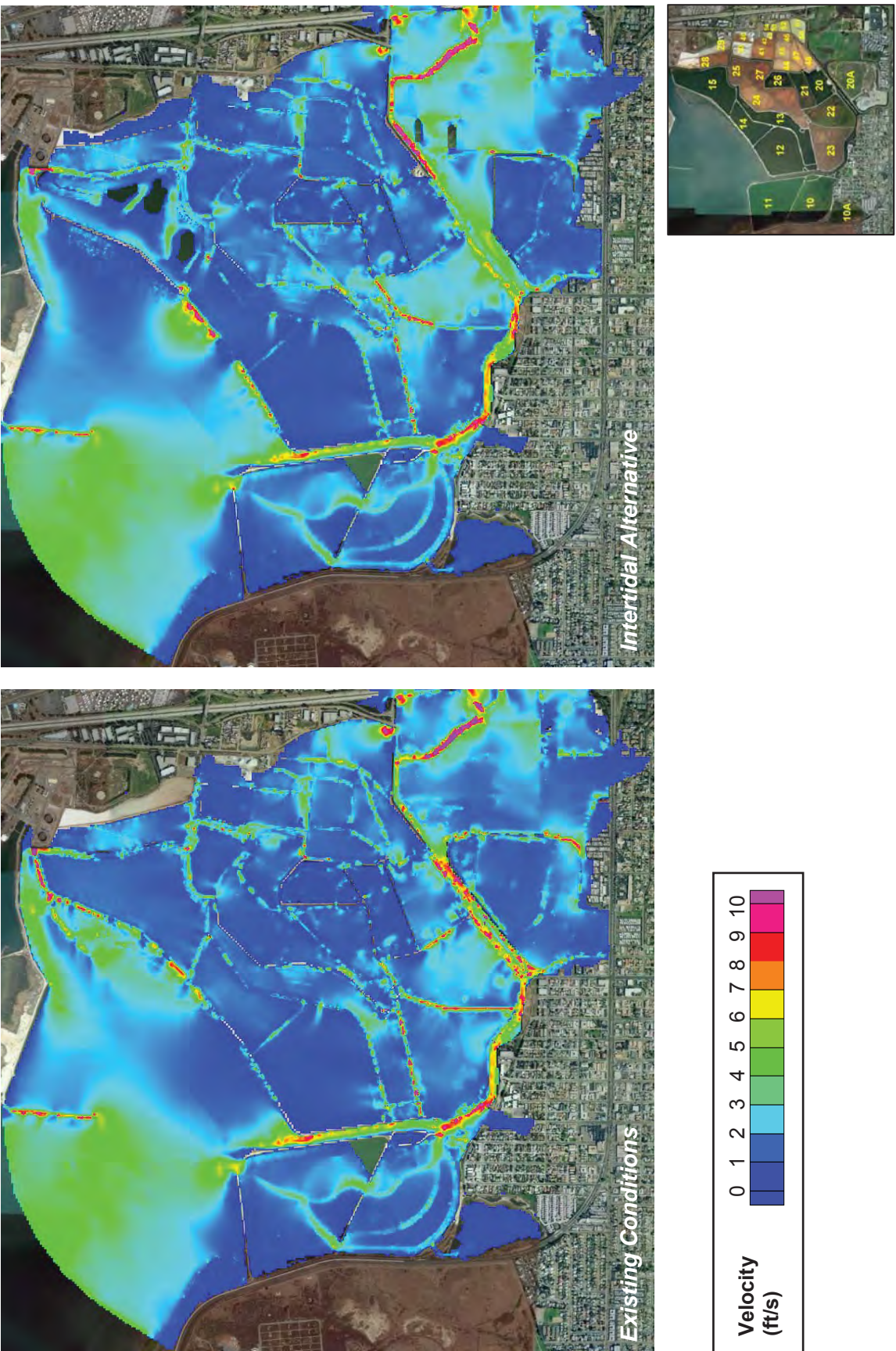
Figure 6.3   Comparison of 100-Year Flood Maximum Velocities for Intertidal Alternative

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6.5

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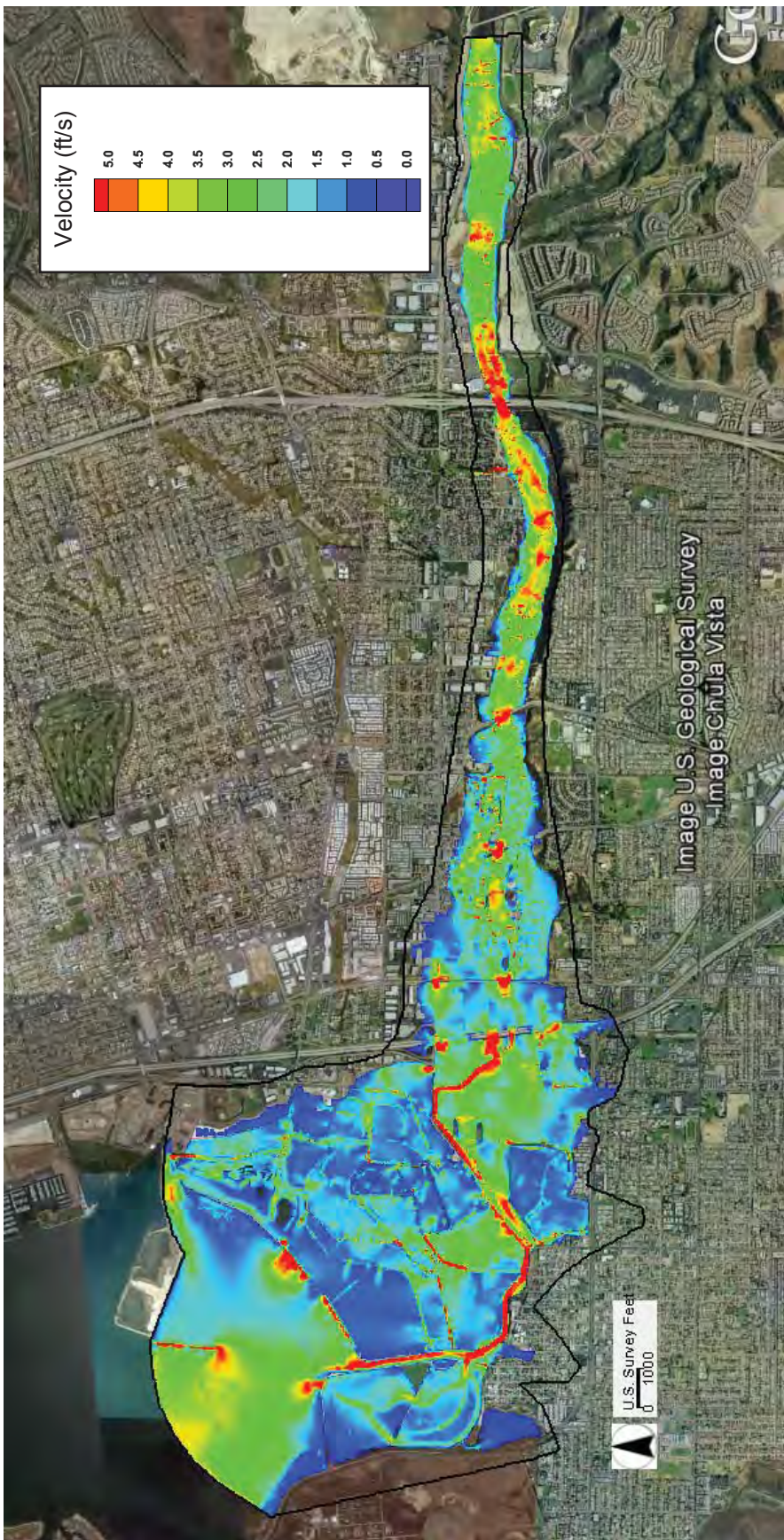
Figure 6.4   100-Year Flood Maximum Velocities for Subtidal Alternative

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6.6

Subtidal Alternative

Existing Conditions

**Velocity
(ft/s)**

0 1 2 3 4 5 6 7 8 9 10
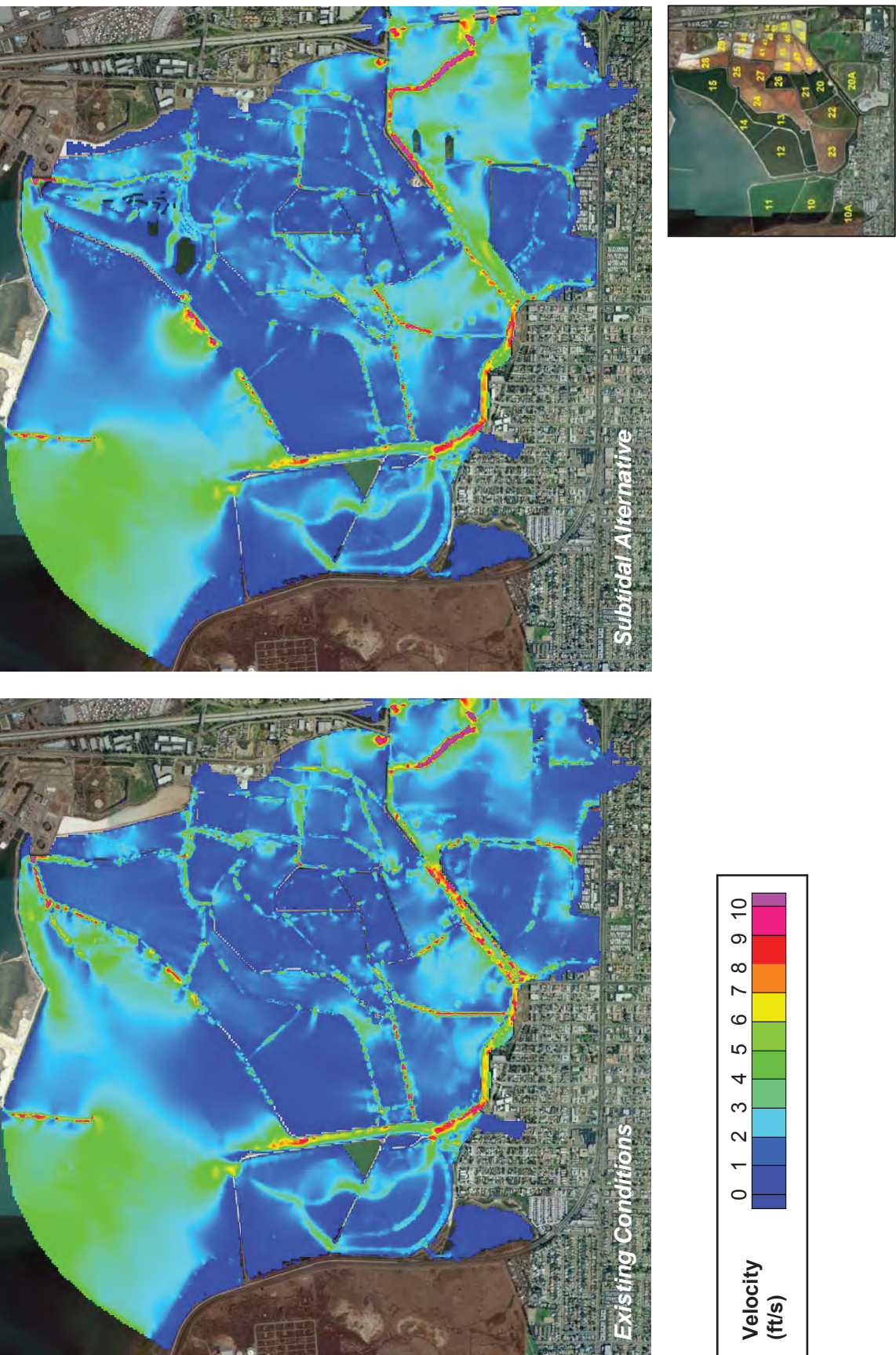
Figure 6.5   Comparison of 100-Year Flood Maximum Velocities for Subtidal Alternative

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6.3   EROSION IMPACTS

To evaluate the erosion impacts, differences in the maximum velocity between the proposed alternatives and Existing Conditions were determined to identify areas with changes in erosion conditions, and the results are shown in Figure 6.6. In the figure, positive values indicate an increase in flood velocities, negative values indicate a decrease in flood velocities, and white-colored areas indicate little to no change in the maximum flood velocities compared to Existing Conditions. As shown in the figure, no changes in flood velocities were determined for the Western Salt Pond Restoration Project (formerly Ponds 10A, 10, and 11). In general, the areas with differences in flood velocities correspond to areas with changes in flood elevations. Blue colored areas indicate lower flood velocities for the proposed alternative compared to Existing Conditions resulting in the reduction of potential erosions. Decreases in flood velocities are found for the river channel along the project area, as well as in Ponds 20 and 15. Lower velocities are also generally observed along the east side of the salt ponds, corresponding to lower flows compared to Existing Conditions. The removal of Pond 15 from the flooded area also results in lower velocities along the Pond 15 levee. Areas with higher velocities for the alternatives are shown by yellow and red areas. Increases in velocity occur due to the redistribution of flows, as previously discussed in Section 5.2. The higher velocities indicate greater potential erosion conditions compared to Existing Conditions, and are further discussed below.

6.3.1   Project Area

As expected, changes in velocity were found throughout the ORF and project areas. Under the alternatives, the existing levee surrounding Pond 20A would be moved to allow tidal flows into the proposed wetland and also enabling flood flows to pass through the project area. Velocities along the existing levee would decrease, as indicated by the blue area surrounding Pond 20A in Figure 6.6. Based on the flood modeling, the proposed alternatives would decrease flood elevations throughout the ORF and project areas due to the expanded flood area, but increase flood velocities compared to Existing Conditions. A soil characterization study conducted for the ORF Site shows that an area east of Nestor Creek (see Figure 6.7) is contaminated with pesticides (Anchor QEA, 2013). Some of these pesticides concentrations reach levels that would be considered hazardous material from the standpoint of waste disposal at a landfill. In addition, some of these pesticides occur in the surface and near surface soils across this area. Consequently, potential project-induced erosion associated with increased flood velocities in this area is of particular concern.

Subtidal Alternative

Intertidal Alternative

Velocity (ft/s)

-4 -3 -2 -1 0 1 2 3 4
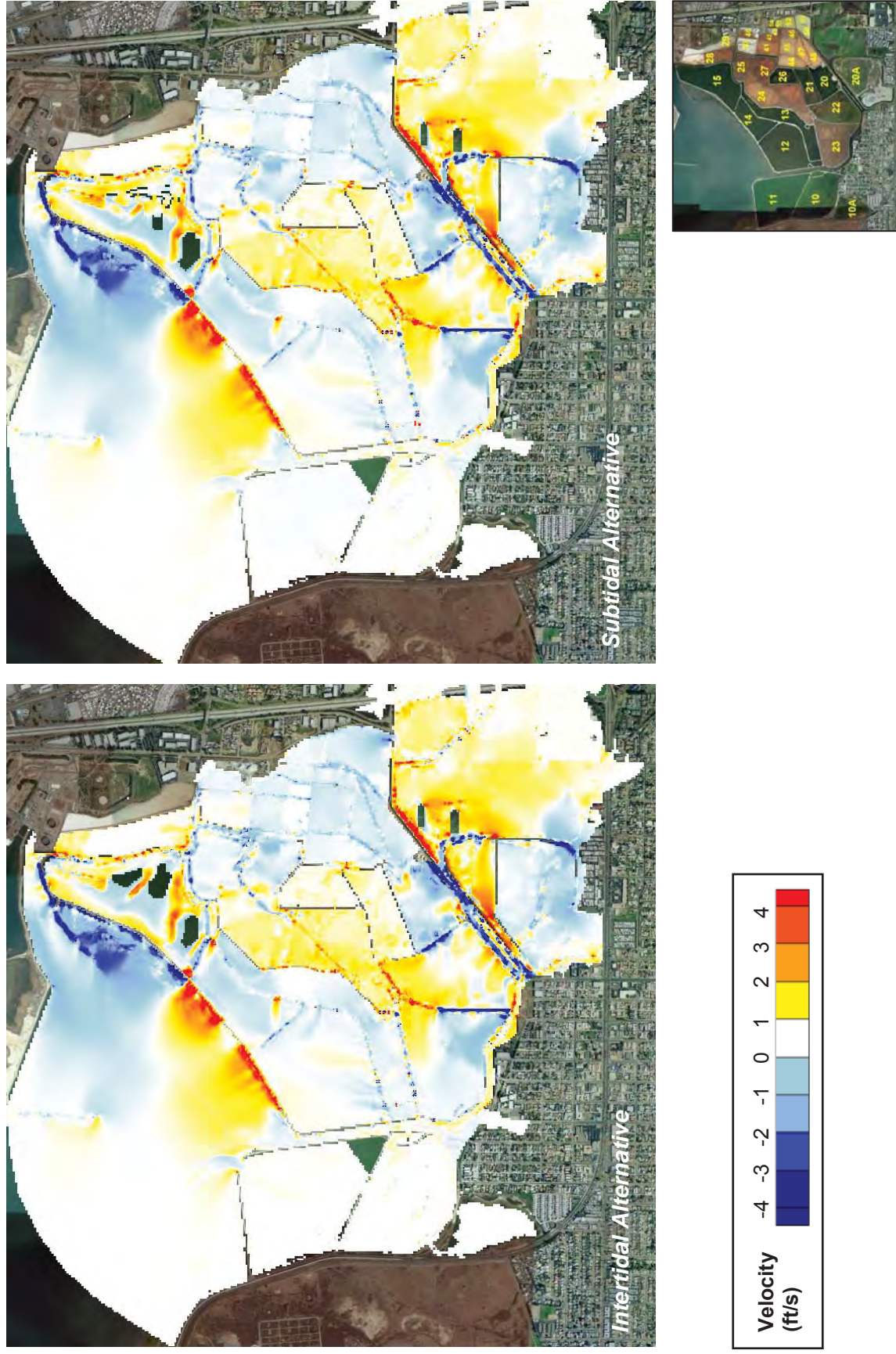
Figure 6.6   100-Year Flood Erosion Impacts based on Maximum Velocities

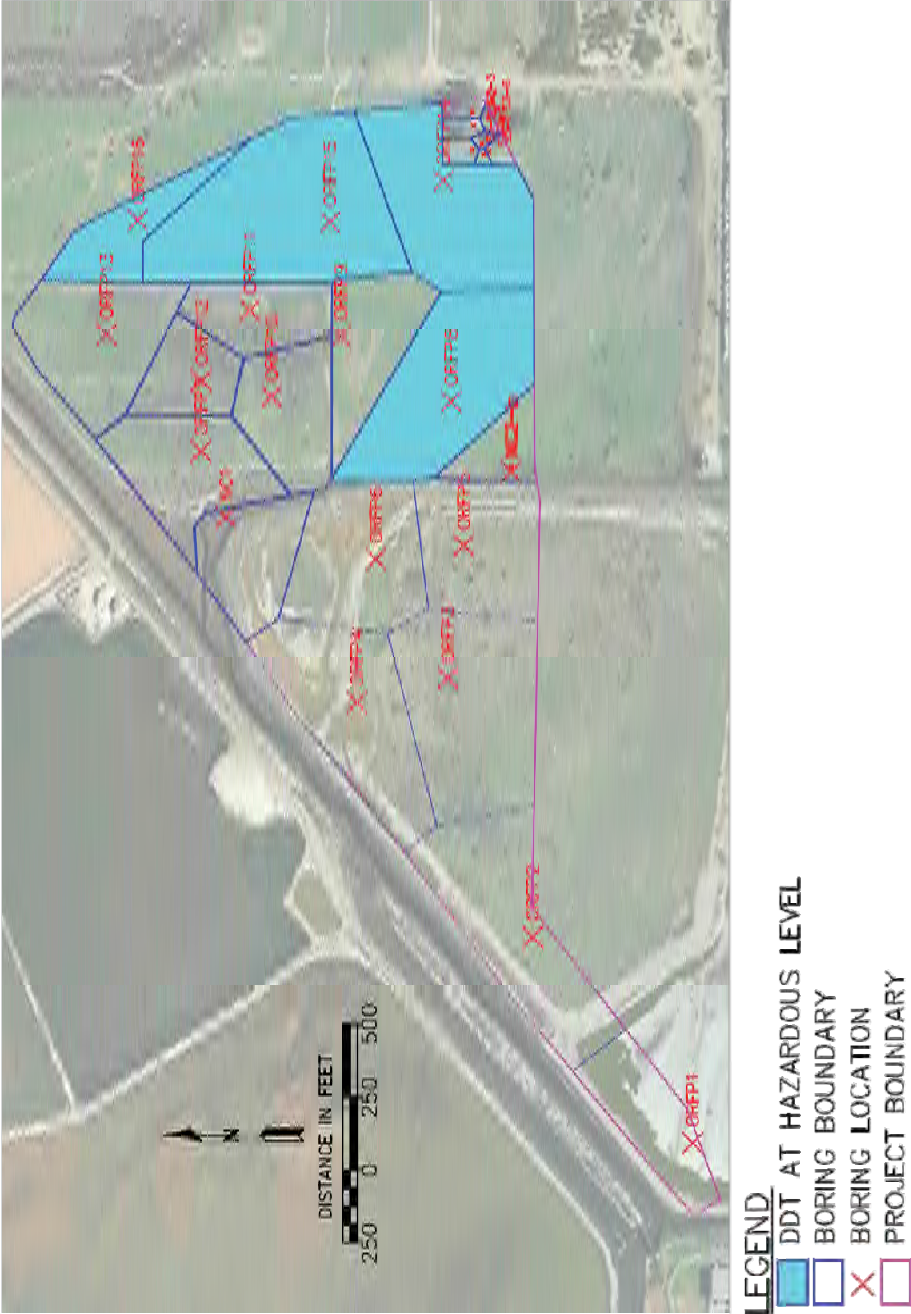*Everest International Consultants, Inc.*

Figure 6.7 Area with DDTs Exceeding CCR22 Hazardous Waste Level

Based on the soil characterization study, the top three feet of the area with pesticide contamination is mainly composed of fine sand to coarse sand. According to the Hjulstrom curve (shown in Figure 6.8), soil composed of fine sand to coarse sand would start to erode when the water velocity reaches and exceeds 0.6 ft/s. To evaluate potential erosion for this area under the 100-year flood, the areas within the ORF Site with maximum flood velocities higher than 0.6 ft/s were identified under Existing Conditions and proposed conditions (Intertidal Alternative and Subtidal Alternative). The areas within the ORF Site with maximum flood velocities higher than 0.6 ft/s under Existing Conditions are shown in Figure 6.9. As shown in the figure, the entire area with contaminated soils would erode under the 100-year flood. Similar plots for areas with maximum flood velocities higher than 0.6 ft/s under the 100-year flood condition for the Subtidal Alternative and Intertidal Alternative are shown in Figure 6.10. As shown in the figure, the maximum flood velocities for the area with contaminated soils are all higher than 0.6 ft/s for both the Subtidal and Intertidal alternatives; hence, similar to Existing Conditions, erosion is likely to occur in this area under the 100-year flood condition. Comparing the flood velocities in this area under the Intertidal Alternative and Subtidal Alternative (Figure 6.10) with those under Existing Conditions (Figure 6.9), the maximum flood velocities in the area would be higher under proposed conditions (Intertidal Alternative and Subtidal Alternative).

The average times when flood velocities at the ORF site are higher than 0.6 ft/s during a 100-year event under Existing Conditions as well as the Intertidal Alternative and Subtidal Alternative were evaluated to assess the potential erosion impact of the proposed alternatives. The times when the flood velocities are higher than 0.6 ft/s were evaluated for seventeen locations at the ORF site (shown in Figure 6.11) by examining the time series of the velocities at each location. An example velocity time series for Location 56 is shown in Figure 6.12. In the figure, the top panel shows a comparison between the velocities under the Intertidal Alternative and Existing Conditions, while the bottom panel shows a comparison between the velocities under the Subtidal Alternative and Existing Conditions. As shown in the figure, the duration for flood velocities higher than 0.6 ft/s under Existing Condition is approximately 10.3 hours, and the corresponding times for the Intertidal Alternative and Subtidal Alternative are approximately the same at about 13.8 hours. The average times when flood velocities are higher than 0.6 ft/s at the ORF site based on an average across the 17 locations shown in Figure 6.11 is approximately 13.2 hours under Existing Conditions, 18.0 hours under the Intertidal Alternative, and 18.2 hours under the Subtidal Alternative. The results indicate that the proposed alternatives have the potential to increase erosion at the ORF site during a 100-year flood event.

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Figure 6.8 Hjulstrom Curve

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Velocity (ft/s)

6.0
5.4
4.8
4.2
3.6
3.0
2.4
1.8
1.2
0.6

Figure 6.9 Area with 100-Year Flood Maximum Velocity Greater than 0.6 ft/s under Existing Conditions

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a) Intertidal Alternative

b) Subtidal Alternative
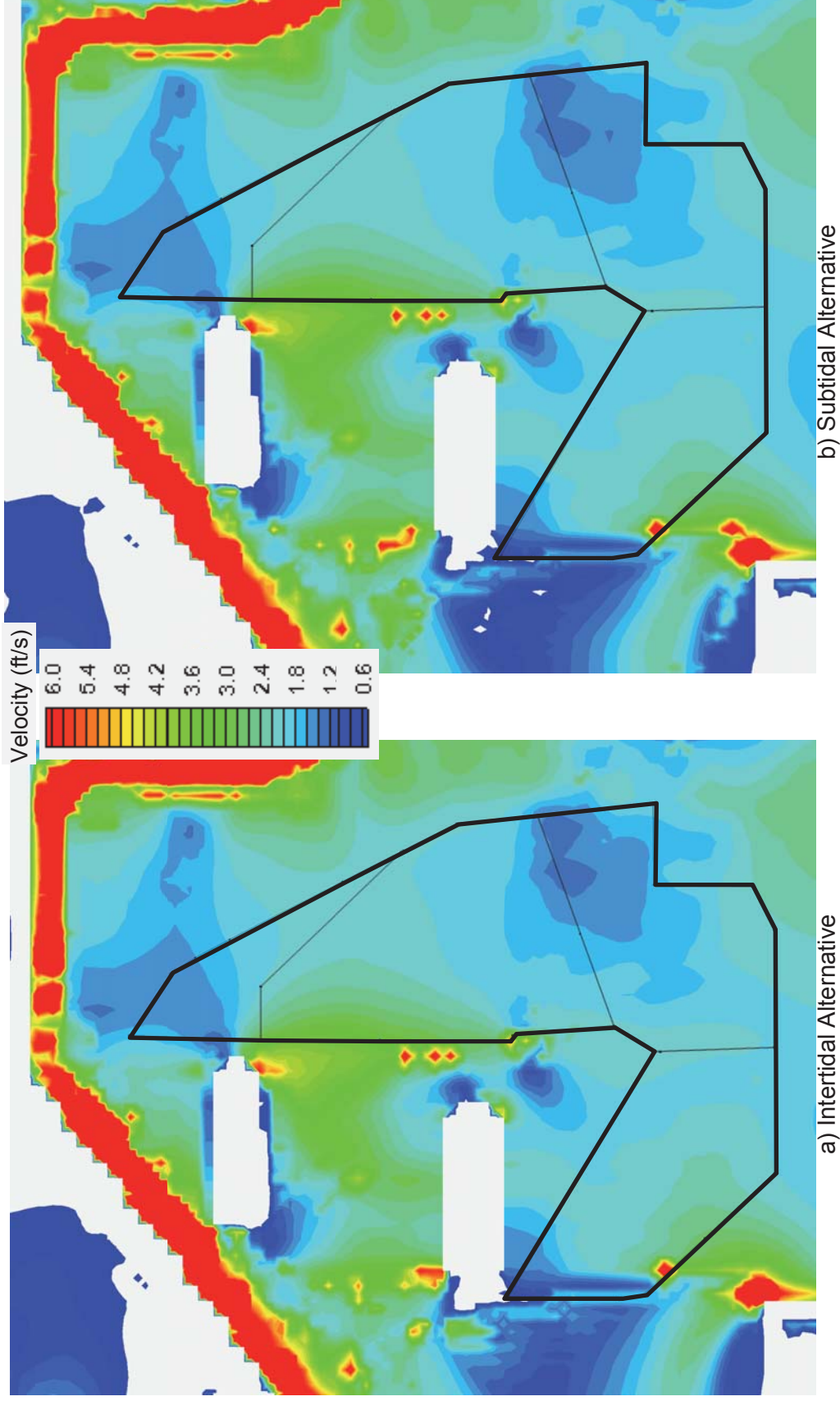
Figure 6.10 Area with 100-Year Flood Maximum Velocity Greater than 0.6 ft/s under Proposed Conditions

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6.14

Figure 6.11 Locations for Calculating the Time Duration when Flood Velocities are Higher than 0.6 ft/s
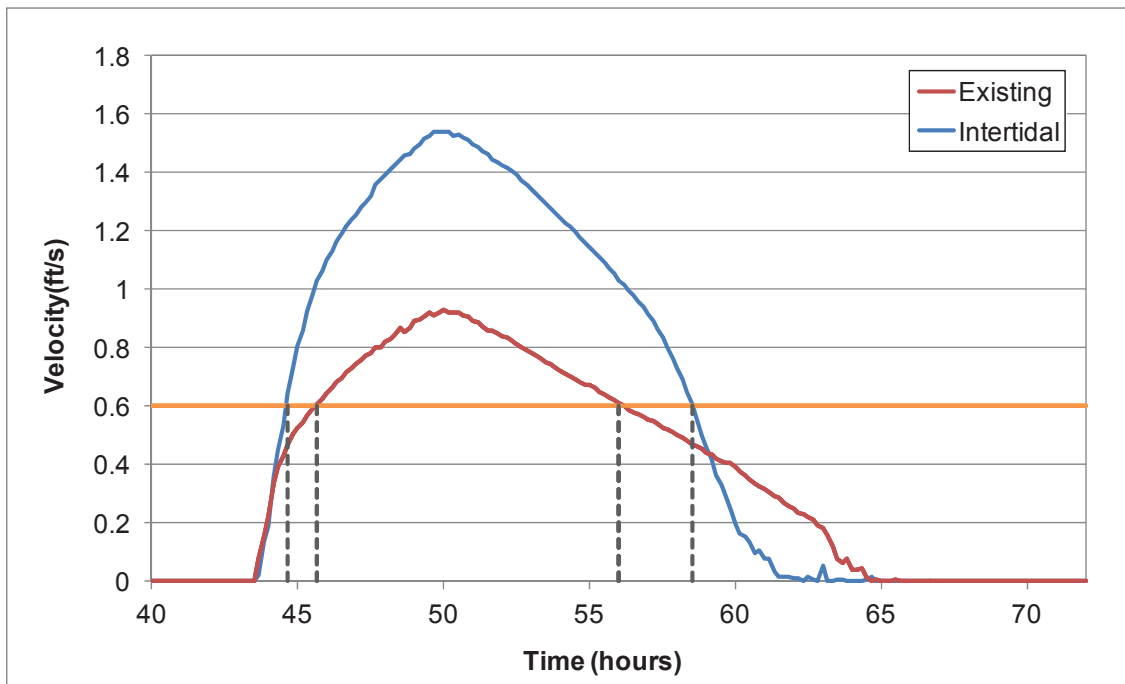
a) Existing versus Intertidal

b) Existing versus Subtidal

Figure 6.12 Velocity Time Series at Location 56

While the proposed alternatives may increase erosion at the ORF site during a 100-year flood event, which by definition will occur once in 100 years (*i.e.,* 1% chance of occurring in any given year), the proposed alternatives may result in less erosion at the ORF site during more frequent smaller flood events (*e.g.,* 5-year flood event and 10-year flood event). The 5-year event will occur on the average once every 5 year (*i.e.,* 20% chance of occurring in any given year), while the 10-year flood event will occur on the average once every 10 years (*i.e.,* 10% chance of occurring in any given year). The areas within the ORF site with maximum flood velocities higher than 0.6 ft/s under Existing Conditions during a 5-year flood event and 10-year flood event are shown in Figure 6.13. As shown in the figure, most of the areas at the ORF Site have flood velocities less than 0.6 ft/s (indicated by the white areas) except along the western edge of the ORF site where flood water from Nestor Creek produces velocities higher than 0.6 ft/s. Similar velocity plots for the Intertidal Alternative and Subtidal Alternative are shown in Figures 6.14 and 6.15, respectively. As shown in the figures, with the proposed alternatives, the maximum velocities are less than 0.6 ft/s for almost the entire ORF site during the 5-year flood event and 10-year flood event. The results indicate that the proposed alternatives have the potential 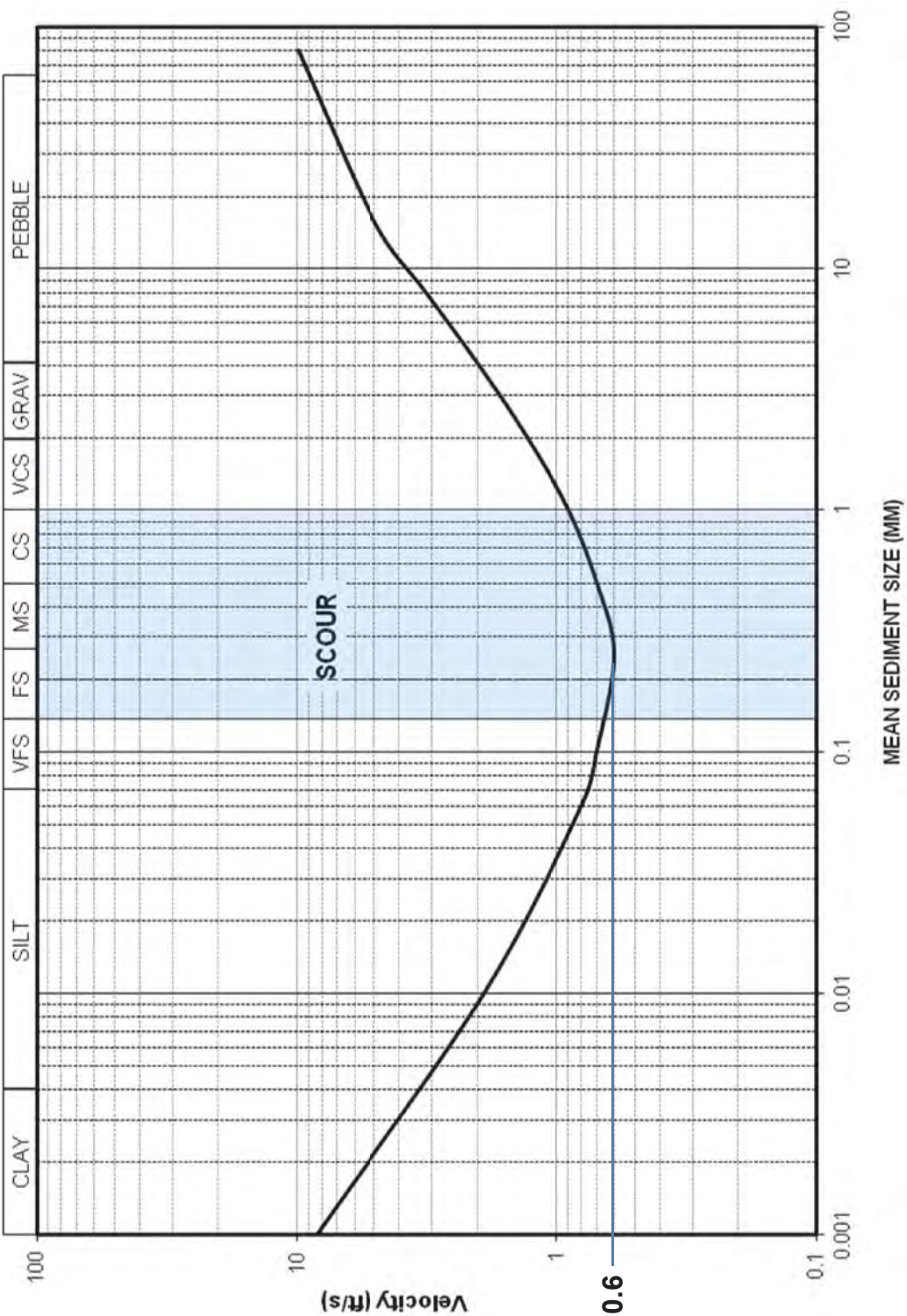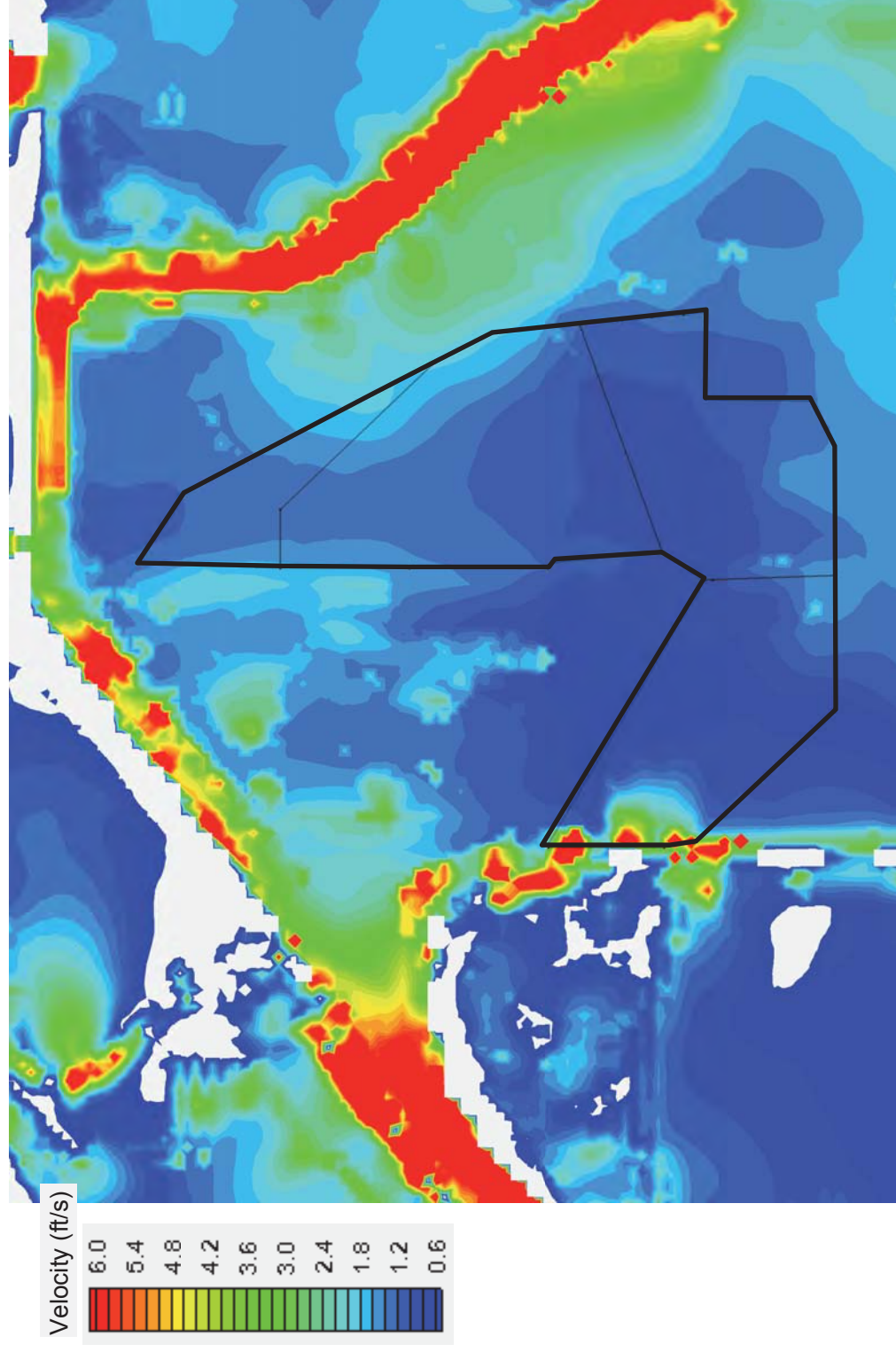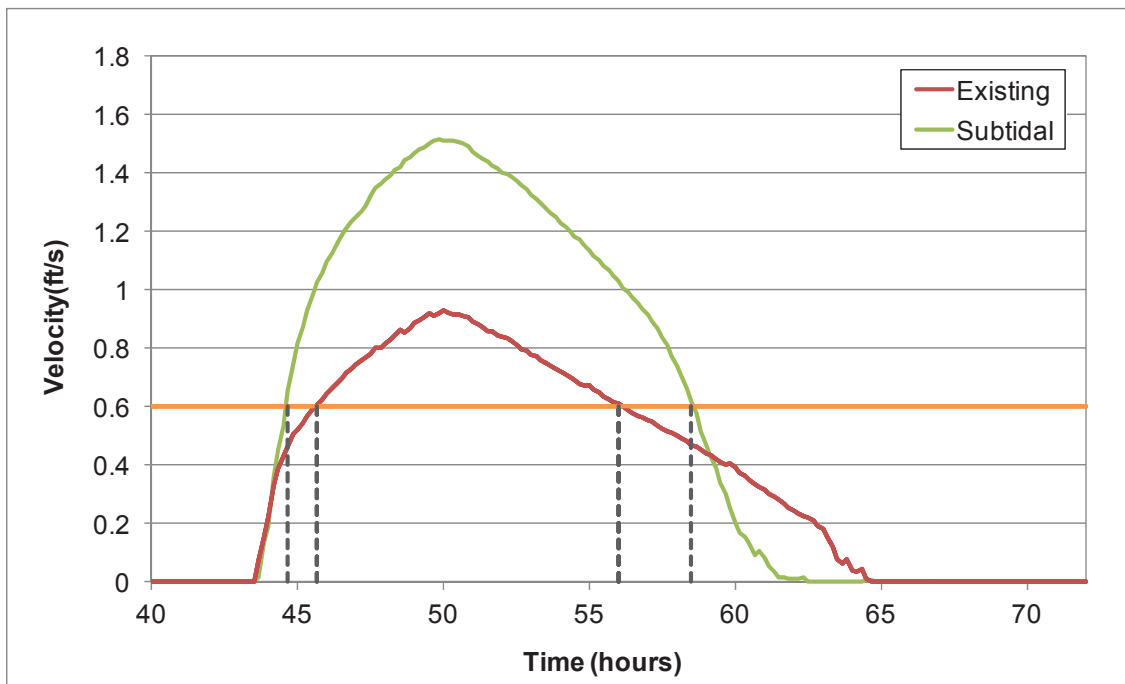to decrease erosion at the ORF site during the more frequent smaller flood events (*e.g.,* 5-year flood event and10-year flood event).

In summary, the proposed Intertidal Alternative and Subtidal Alternative would potentially increase erosion at the ORF site during the 100-flood event, but would potentially decrease erosion during the more frequent, smaller flood events (*e.g.,* 5-year flood event and 10-year flood event). The overall potential impact of the proposed alternatives on erosion across the ORF site is likely to be small to negligible across the range of flood events that would be expected to occur within a 100-year period.

If it is desired to minimize the potential erosion impact of the proposed alternatives across the ORF site during a 100-year flood event, vegetation can be planted at the site to increase friction (*i.e.,* Manning's coefficient) to slow down the flow. In addition to increasing friction, the roots of the vegetation would help bind the soil, making it less erodible (*i.e.,* the flood velocity for initiation of erosion would become higher than 0.6 ft/s). It was determined that if the Manning's coefficient is increased to 0.15 from the existing of 0.05 for the ORF site, the flood velocities for the proposed alternatives would be slowed down to approximately the same as those under Existing Conditions. Figure 6.16 shows the differences in maximum flood velocities between the proposed Intertidal Alternative and Subtidal Alternative with vegetation (*i.e.,* Manning's Coefficient of 0.15) and Existing Conditions. It can be seen from the figure that with the higher Manning's coefficient, the maximum flood velocities with the proposed alternatives are lower than those under Existing Conditions (the blue areas in the figure) for the eastern portion of the ORF site. For the area adjacent to Nestor Creek, the maximum velocities under the proposed alternatives would still be slightly higher (the yellow areas in Figure 6.16) than those under Existing Conditions. In general, the areas with reduced velocities (blue) are slightly larger than the areas with increased velocities (yellow). Hence, the erosion conditions at the ORF site

Velocity (ft/s)

6.0
5.4
4.8
4.2
3.6
3.0
2.4
1.8
1.2
0.6

b) 10-Year Velocity

a) 5-Year Velocity
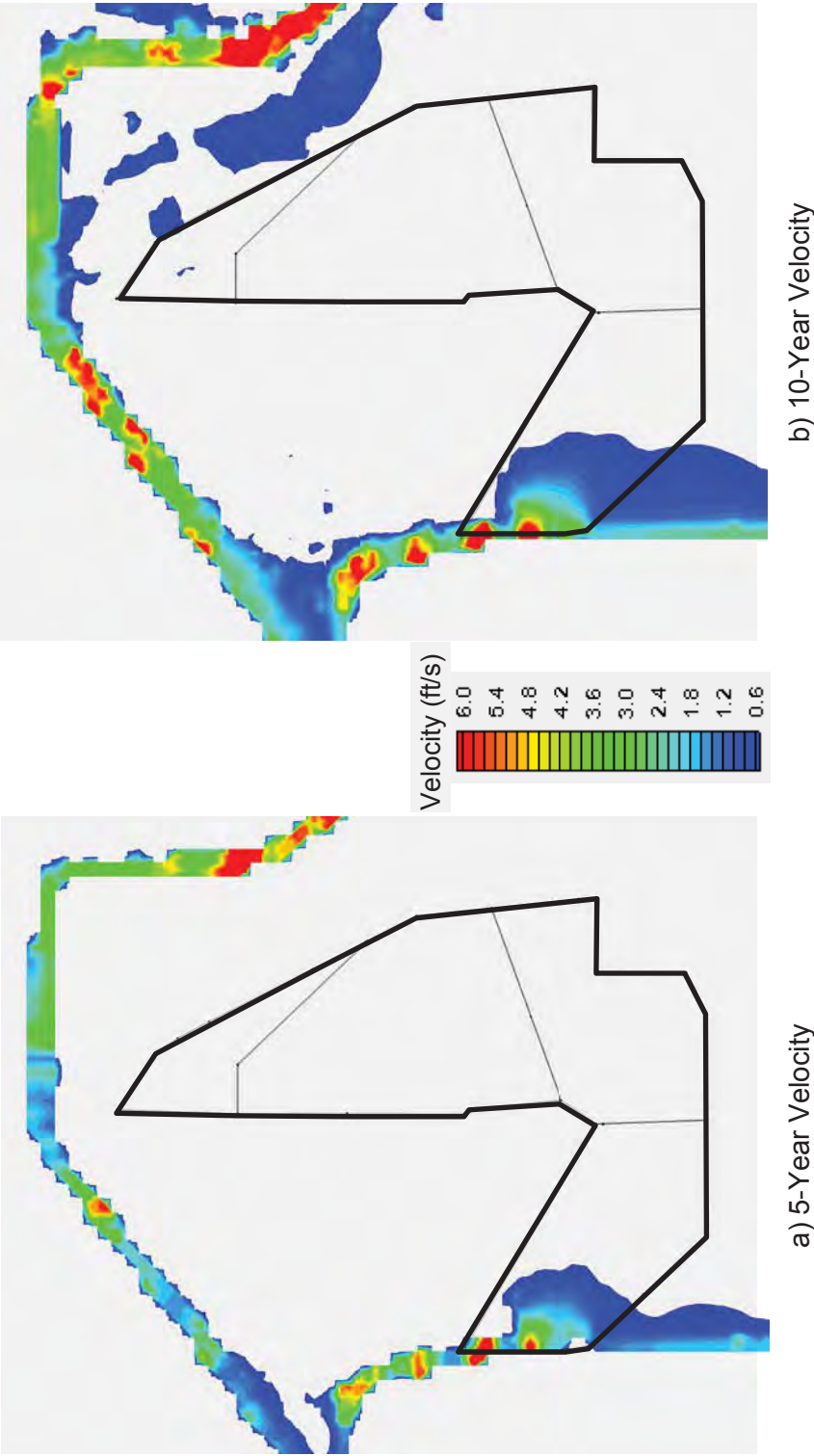
Figure 6.13 Maximum Velocity of ORF Site under Existing Conditions

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Velocity (ft/s)
6.0 5.4 4.8 4.2 3.6 3.0 2.4 1.8 1.2 0.6

a) 5-Year Velocity

b) 10-Year Velocity
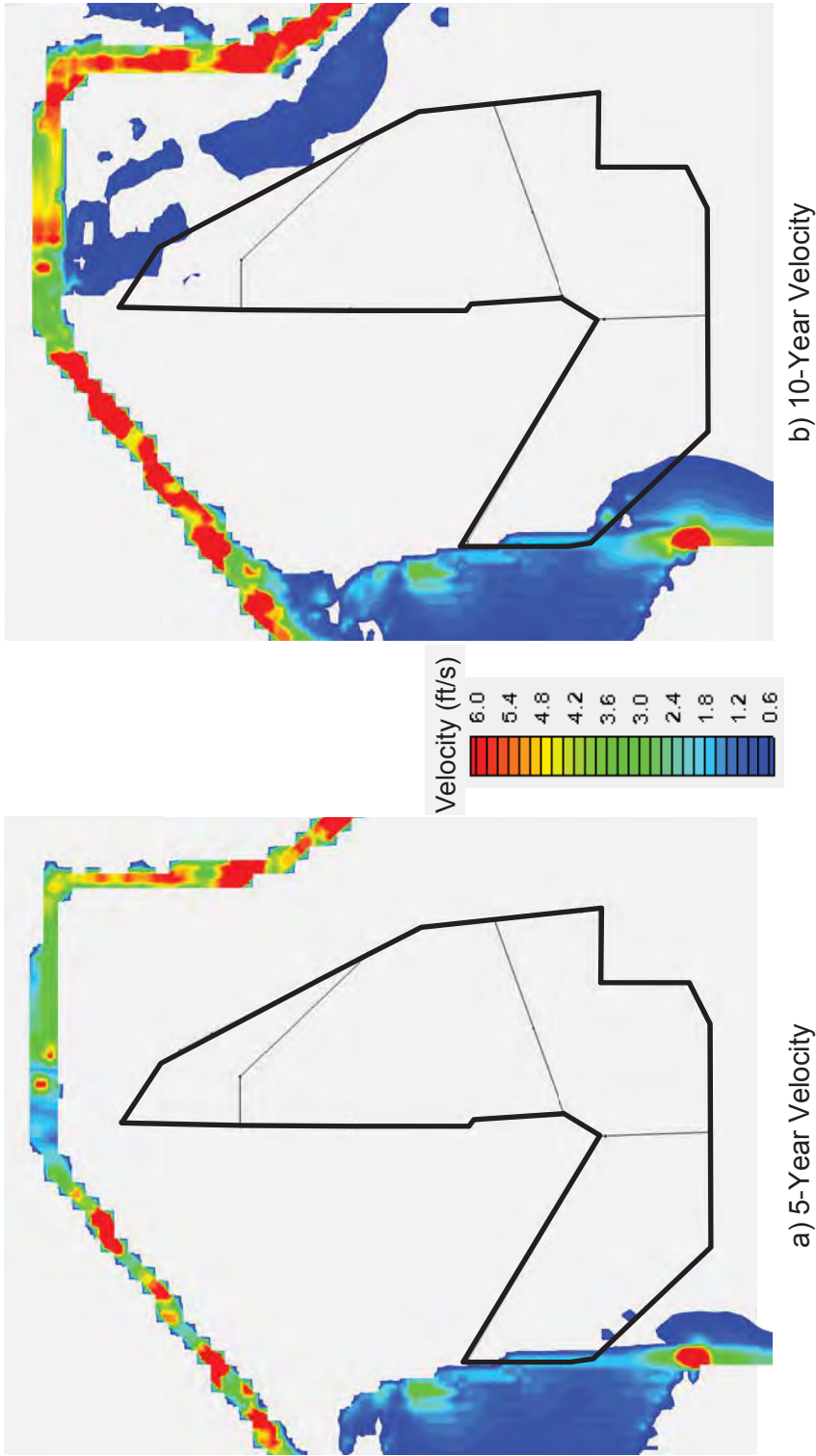
Figure 6.14 Maximum Velocity of ORF Site under Intertidal Alternative Conditions

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6.19

Velocity (ft/s)

6.0
5.4
4.8
4.2
3.6
3.0
2.4
1.8
1.2
0.6

a) 5-Year Velocity

b) 10-Year Velocity
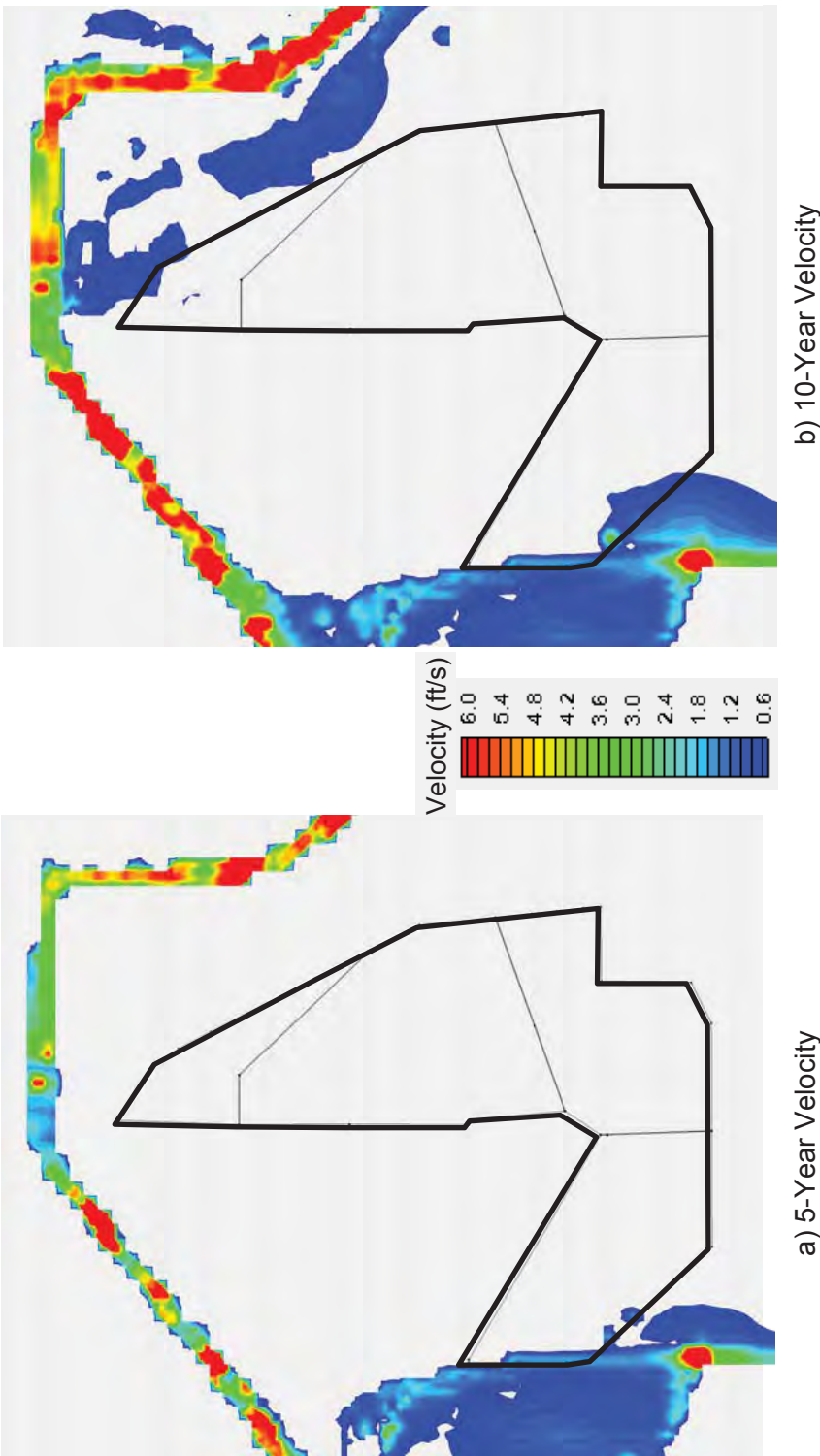
Figure 6.15 Maximum Velocity of ORF Site under Subtidal Alternative Conditions

Velocity
Diff. (ft/s)

3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0

a) Intertidal Alt. - Existing
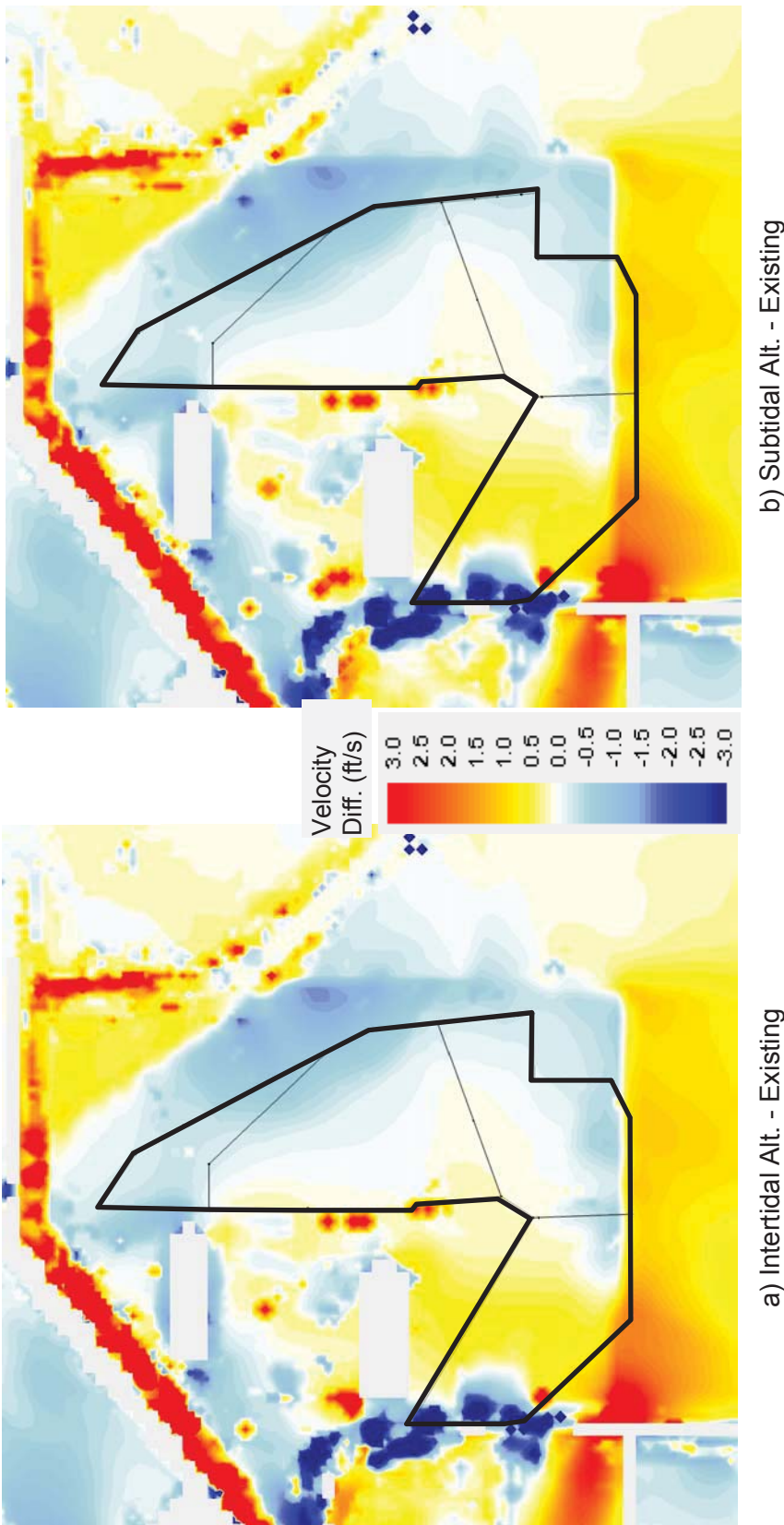
b) Subtidal Alt. - Existing

Figure 6.16 Difference in Maximum Velocity Between Proposed Conditions and Existing Conditions for a 100-Year Flood Event

under the 100-year flood event are likely to be similar between the proposed alternatives with vegetation and Existing Conditions.

Besides the project area, other areas with observable change in velocities include the areas along the bike path, Pond 15, and the levees along Ponds 12 and 14. Changes in velocity at these locations are further described below.

6.3.2   Bike Path

The proposed alternatives would result in changes to the flood velocities in the river channel along the bike path. A closer view of the differences in the maximum velocity along the bike path is provided in Figure 6.17. The northern portion of the bike path along Pond 48 is not flooded (i.e., flows do not overtop the bike path) under Existing or proposed conditions. In this area, higher velocities occur in the river channel along the bike path, while lower velocities occur in the river channel along the center and south portions of the bike path. Water elevations and velocities along the river channel near Pond 48 (Location A) for Existing Conditions, Intertidal Alternative, and Subtidal Alternative are compared in Figure 6.18. In the top panel, water elevations for the Intertidal (orange line) and Subtidal Alternative (green line) are lower compared to Existing Conditions (blue line). The corresponding velocities for the proposed alternatives, shown in the bottom panel, are higher than Existing Conditions due to the reduced water depth. These higher velocities in the river channel are similar in magnitude to velocities that occur at the center and south ends (adjacent to Pond 20 and 22) under Existing Conditions. In general, erosion impacts to the river channel along the north end of the bike path will be similar to erosion conditions currently occurring along the river channel at the south end of the bike path. In other words, the proposed alternatives would essentially shift the higher velocities along the river channel from the south portion of the bike path to the north portion of the bike path. A rock revetment could be placed along the northern portion of the bike path to prevent potential bank erosion caused by the increase in flood velocities associated with the proposed alternatives. Based on the 100-year flood velocities, a one-foot layer of 5-inch (D_{50}) rock would be sufficient to prevent erosion along the bank. A typical cross-section for this revetment and the proposed location are both shown in Figure 6.19.

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Subtidal Alternative

Intertidal Alternative

Velocity (ft/s)

-4 -3 -2 -1 0 1 2 3 4
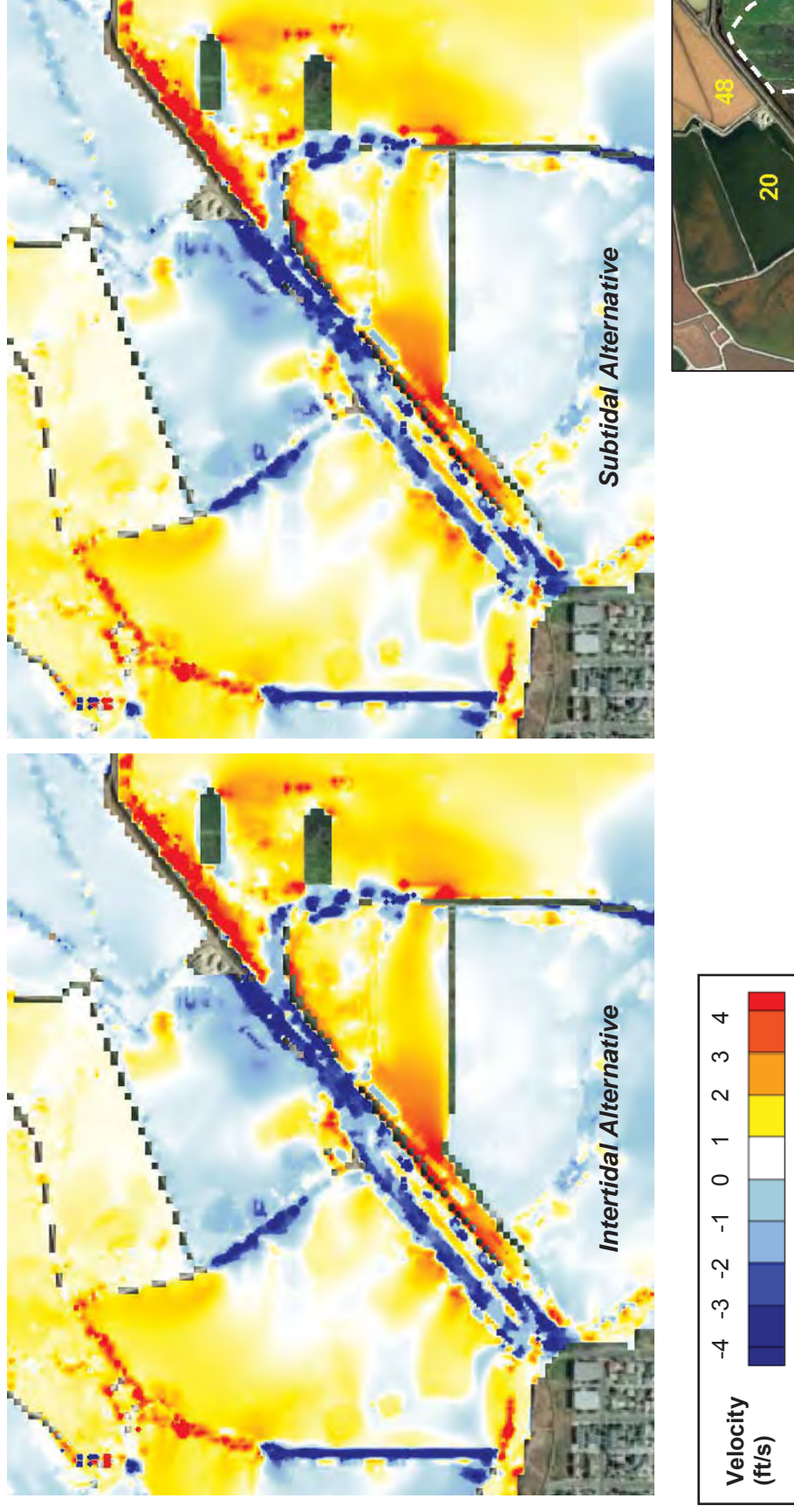
Figure 6.17   100-Year Flood Erosion Impacts along Bike Path

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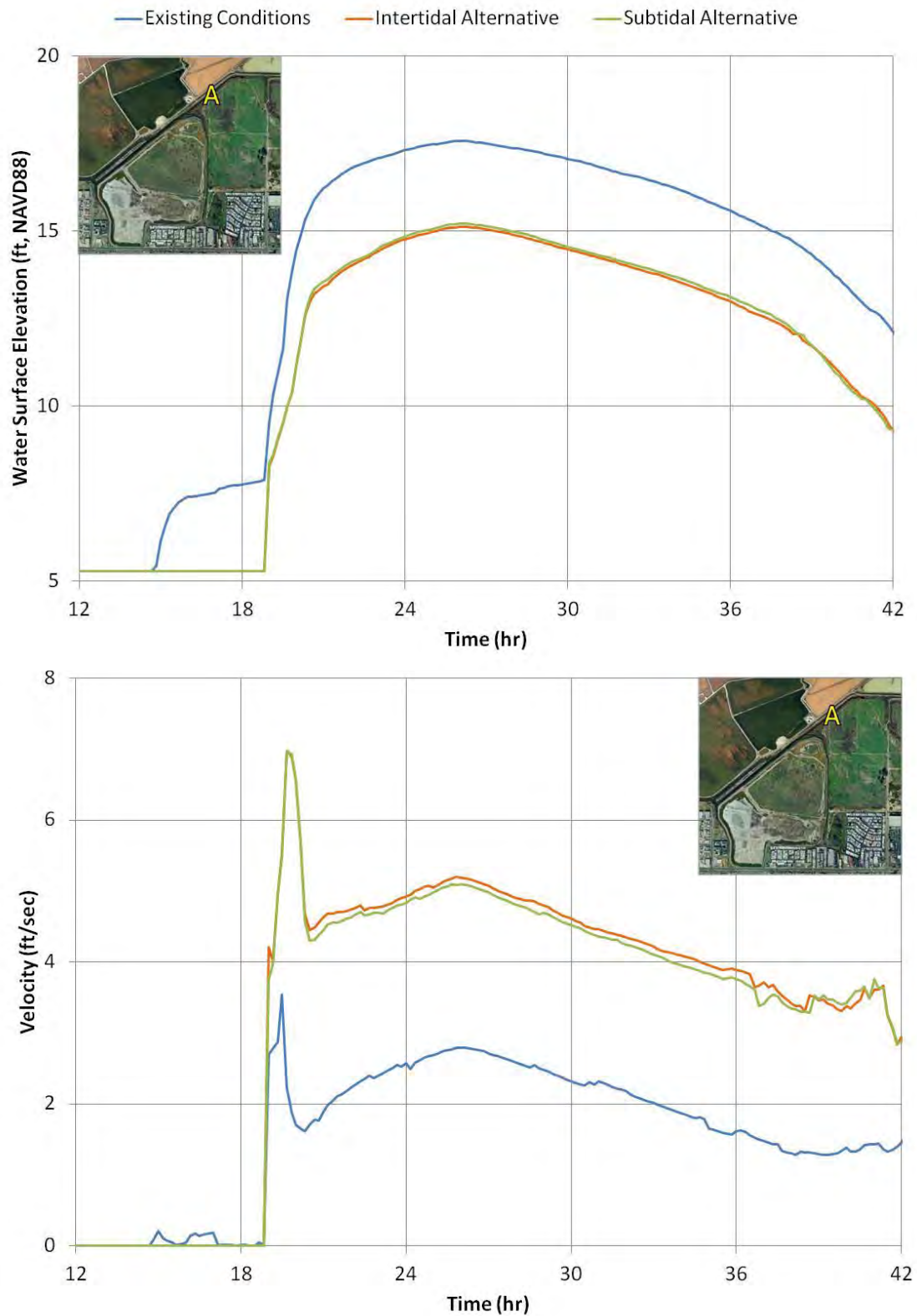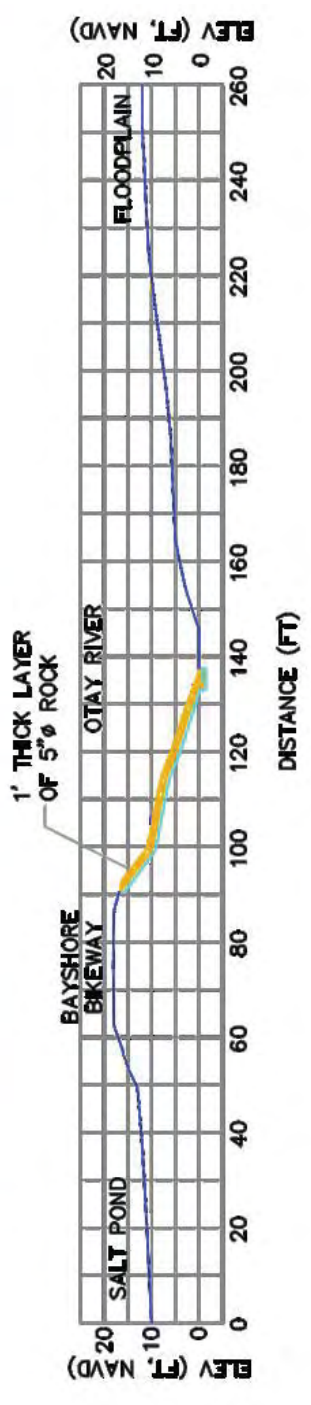
Figure 6.18   100-Year Flood Water Elevation and Velocity Comparisons at Location A

PLAN

SECTION A–A

Figure 6.19 Proposed Revetment Location and Typical Cross-section

Along the central portion of the bike path along Pond 20, the bike path is overtopped into Pond 20 during the 100-year flood. Under the proposed alternatives, both flood elevations and velocities along the river channel and bike path are lower compared to Existing Conditions. Hence, no erosion impacts are anticipated in this stretch of the river channel or bike path.

Along the southern portion of the bike path, lower velocities in the river channel would occur under the proposed alternatives as compared to Existing Conditions. However, higher flood velocities were shown to occur along the bike path and Pond 22 levee. These higher velocities are attributed to the increase in flow over the bike path and levee. Water elevations and velocities of the flows overtopping the bike path (Location B) are compared in Figure 6.20. As shown in the figure, the increase I flow in this area would result in higher water elevations and velocities for both the Intertidal (orange line) and Subtidal Alternatives (green line) as compared to Existing Conditions (blue line).

6.3.3   Ponds 12, 14 and 15

Modifications to Pond 15 under the proposed alternatives would result in changes in velocities surrounding Pond 15, as illustrated in Figure 6.21. Differences in velocities occur along the levee separating the salt ponds and San Diego Bay. Lower velocities are shown along Pond 15 since flood flow no longer overtops the levees at Pond 15. Higher velocities are apparent along the levees of Ponds 12 and 14 due to higher flows overtopping of the levees into San Diego Bay. Under the proposed alternatives, flood elevations in Ponds 12 and 14 are higher due to the redistribution of flows through the salt ponds and diversion of flows around Pond 15, as previously discussed in Section 5.2.4. The higher flood elevations would cause an increase in flows and velocities across the levees into San Diego Bay. The proposed alternatives effectively shift the higher velocities due to flows overtopping the levee from Pond 15 to Ponds 12 and 14. The higher velocities may increase erosion along the levees for Ponds 12 and 14.

The isolation of Pond 15 from flood flows also increases flows into Ponds 28 and 29. Differences in velocities occur along the east side of Pond 15, as shown in Figure 6.21. Decreases in velocity are observed along the Pond 15 levee due to the raising of the levee under the proposed alternatives. Increases in velocity are shown to occur along the channel between Pond 15 and 28. However, the range in velocities occurring along this channel is the same for existing and proposed conditions, as shown previously in Figures 6.3 and 6.5. Erosion impacts along the channel between Ponds 15 and 28 are likely to be localized scour in limited areas of the channel. Increases in flow into Ponds 28 and 29 also result in higher velocities within the ponds, which may result in localized scour where flows overtop the levee.
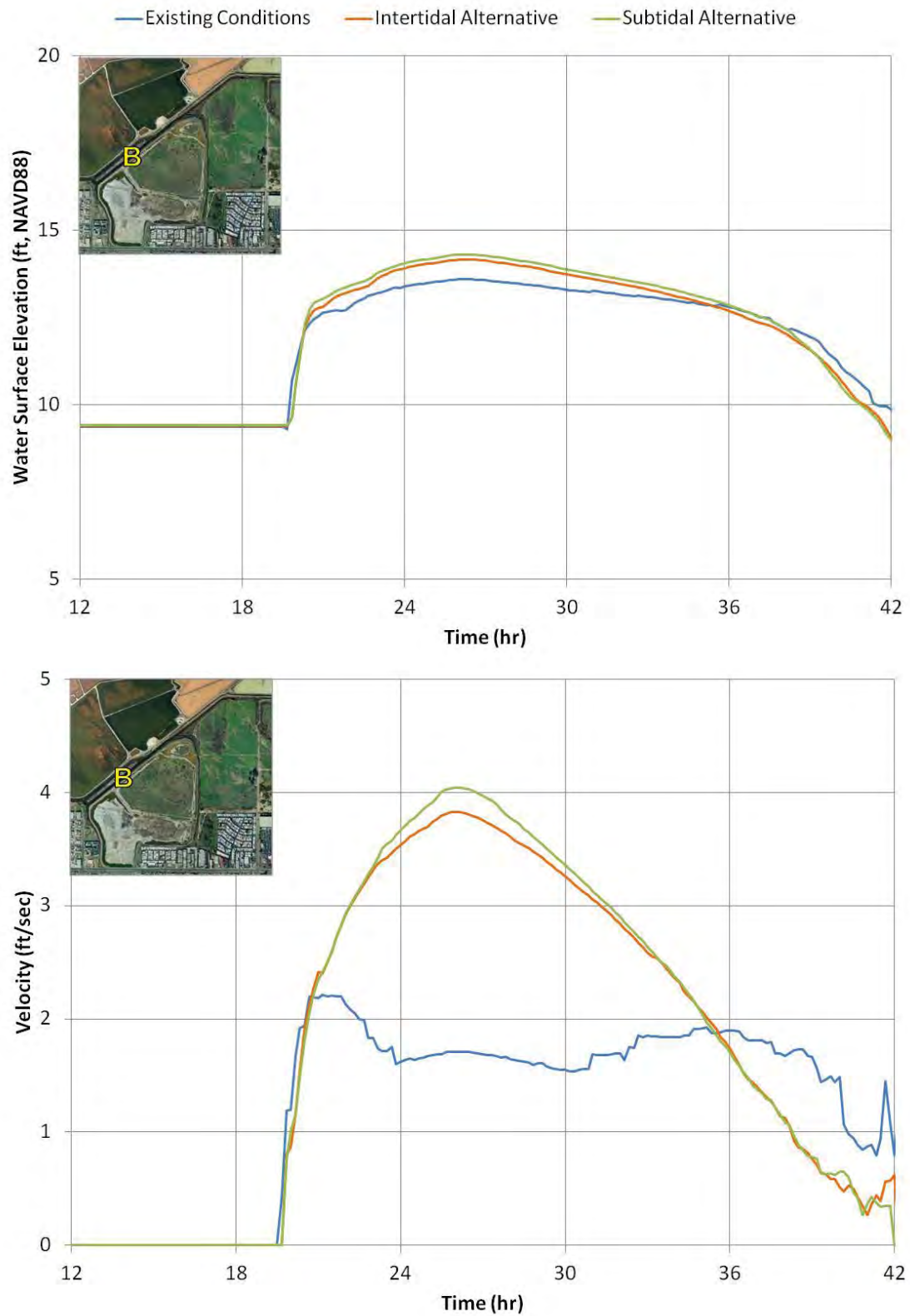
Figure 6.20   100-Year Flood Water Elevation and Velocity Comparisons at Location B

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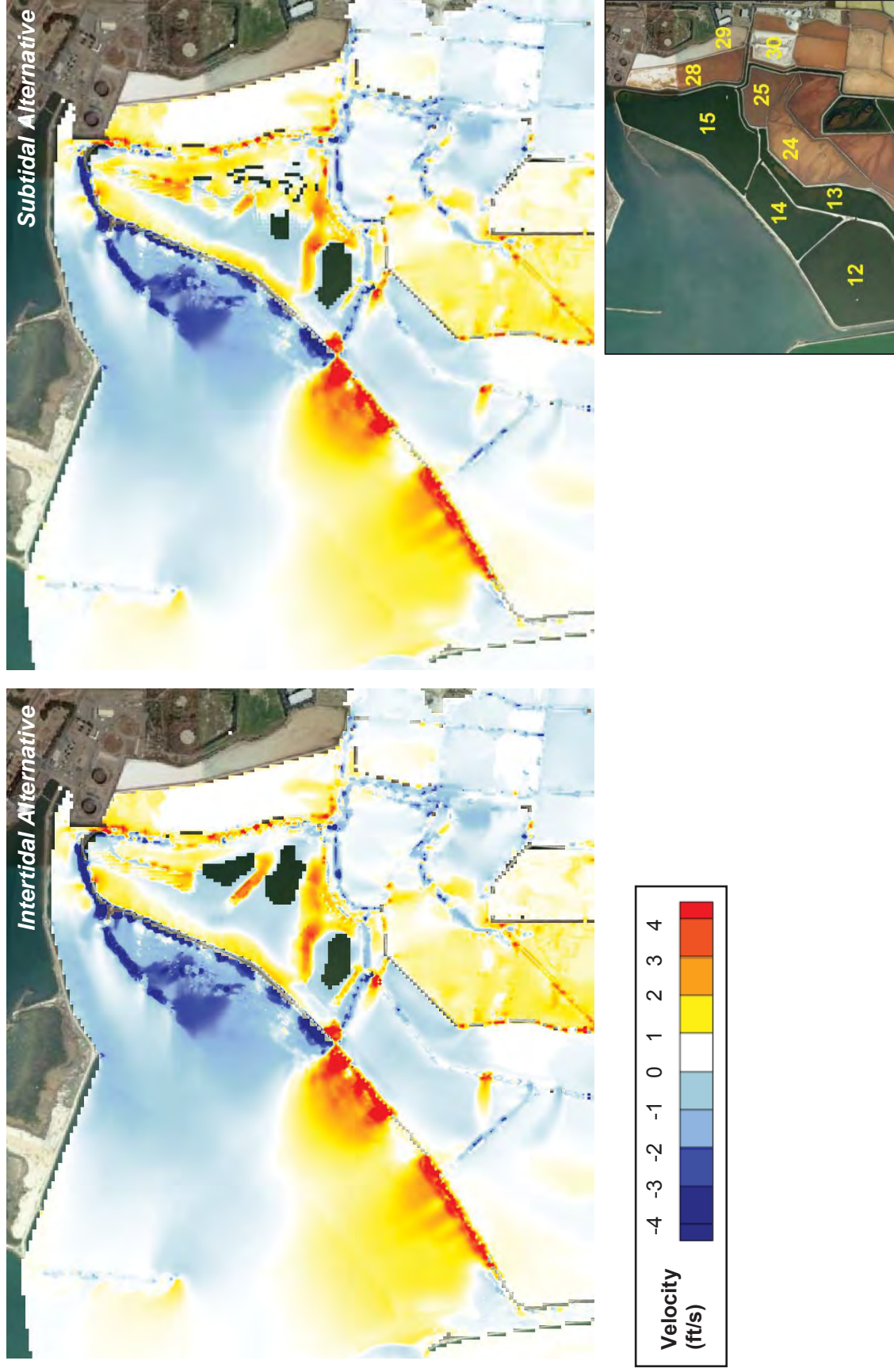F luvial Hydraulics Study Report

Subtidal Alternative

Intertidal Alternative

Velocity
(ft/s)

-4 -3 -2 -1 0 1 2 3 4

Figure 6.21   100-Year Flood Erosion Impacts near Pond 15

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6.28

6.4   SUMMARY

Hydrodynamic modeling was conducted to establish flood velocities for Existing Conditions, Intertidal Alternative, and Subtidal Alternative. In general, the highest velocities occur along the river channel, as well as across levees when flow overtopping occurs. Under the proposed alternatives, the velocity distribution in the ORF (ORF) would be altered due to distribution of flood flows through the proposed alternatives. Differences in the maximum flood velocities between proposed alternatives and Existing Conditions were used to characterize and qualitatively evaluate potential erosion impacts. Areas with the same or lower velocities than Existing Conditions are expected to have no erosion impacts, while areas with higher velocities may have erosion impacts.

For most of the ORF, the proposed alternatives will not change flood velocities, including tidally influence areas such as the Western Salt Pond Restoration Project (formerly Ponds 10A, 10, and 11). In general, no erosion impacts are expected within most of the salt ponds. Decreases in erosion impacts were determined for Ponds 20 and Pond 15, as well the central portion of the bike path and river channel adjacent to the proposed wetland. Minor reductions in erosion impacts were determined for several ponds on the eastern side of the salt ponds.

The proposed alternatives divert flood flows through the project area resulting in higher velocities along the northern portion of the bike path. A rock revetment could be placed along this portion of the bike path embankment to mitigate the potential increase in bank erosion. High velocities were also predicted in areas east of Nestor Creek that contain highly contaminated soils. Based on the soil characteristics for the area, the soils are likely to erode under existing conditions during a 100-year flood.

The proposed alternatives will also result in erosion impacts along the levees separating the salt ponds and San Diego Bay. Under the proposed alternatives, Pond 15 would be isolated from the flood area, thereby increasing flood elevations in Ponds 12, 14 and 28. The overtopping of Pond 15 into San Diego Bay under Existing Conditions will be diverted to Ponds 12 and 14. The overtopping of the Pond 12 and 14 levees into San Diego Bay may increase erosion along the levees. Higher velocities that may result in localized scour were also determined along the levee between Ponds 15 and 28.

7. FLUVIAL SEDIMENTATION ANALYSIS

7.1 APPROACH

A fluvial sedimentation analysis was conducted to evaluate fluvial sediment delivery from the upstream watershed to the proposed wetlands and the subsequent rate of sedimentation in the proposed wetlands. Sedimentation at the mouth of the Otay River (where the Otay River meets San Diego Bay) associated with coastal processes (*e.g.,* tides, waves) is addressed in a companion study (Jenkins, 2014). The approach for the fluvial sedimentation analysis is summarized below.

1. Estimate the annual sediment loadings from the Otay River Watershed
2. Evaluate the portion of sediment loadings from the Otay River Watershed that will get to the proposed wetland area
3. Estimate the annual sedimentation rate in the wetland based on the portion of the sediment entering the wetland that is likely to settle in the wetland

7.2 SEDIMENT LOADING FROM OTAY RIVER WATERSHED

In the Otay River Watershed, fluvial sediments are transported from the watershed along the Otay River into San Diego Bay. Soils along mountains and canyons are primarily eroded during storm events and washed downstream. A portion of eroded sediment, typically gravels and sands, deposits along the river bed, while finer sediment generally deposits within the river floodplain or delta that forms where the river meets San Diego Bay.

Historically, the downstream end of the Otay River was a typical river delta with multiple pathways to San Diego Bay. This "natural" river configuration was altered by the channelization of the river that has occurred through the construction of dikes and levees for salt ponds and agriculture practices (Aspen 2006 and River Partners 2008). Changes in the river configuration are illustrated in Figure 7.1 based on a comparison of the river in 1903 and 1941. In the figure, the historical delta features are indicated by the magenta lines showing the river planform circa 1903. The river floodplain contained multiple meandering channels leading to San Diego Bay. The channelized river is shown by the green lines based on the river alignment circa 1941. The river pathway has been confined along the edges of the floodplain, which is similar to the current condition of the river. Changes to the lower end of the Otay River are also illustrated by maps of the floodplain area circa 1902, 1930, 1950, and 1978 provided in Figure 7.2. The earlier maps circa 1902 and 1930 show the delta formation along San Diego Bay. The 1950 and 1978 maps show salt ponds and/or agricultural areas in the former river delta.
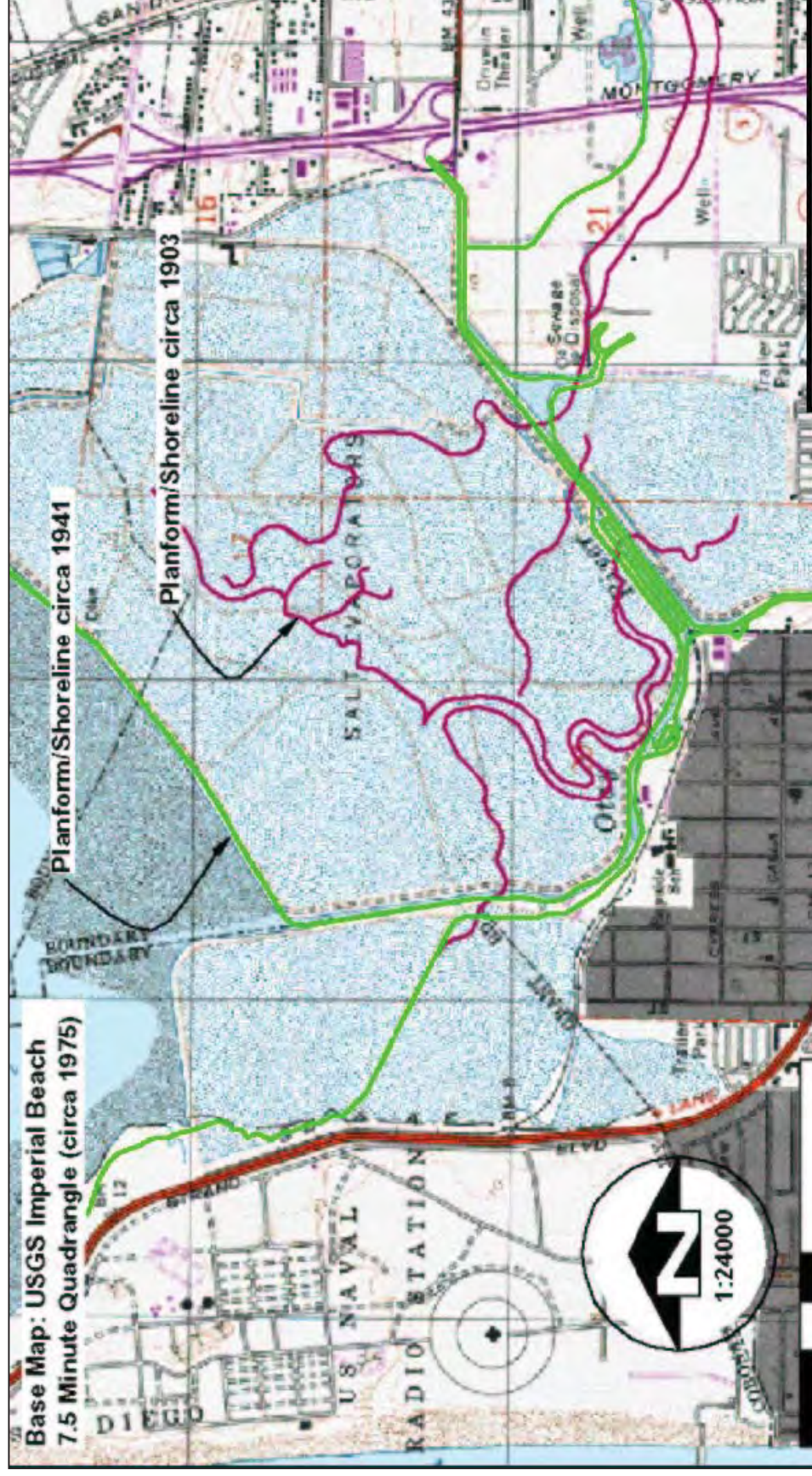
Source: Aspen 2006

Figure 7.1   Historical Changes to the Otay River
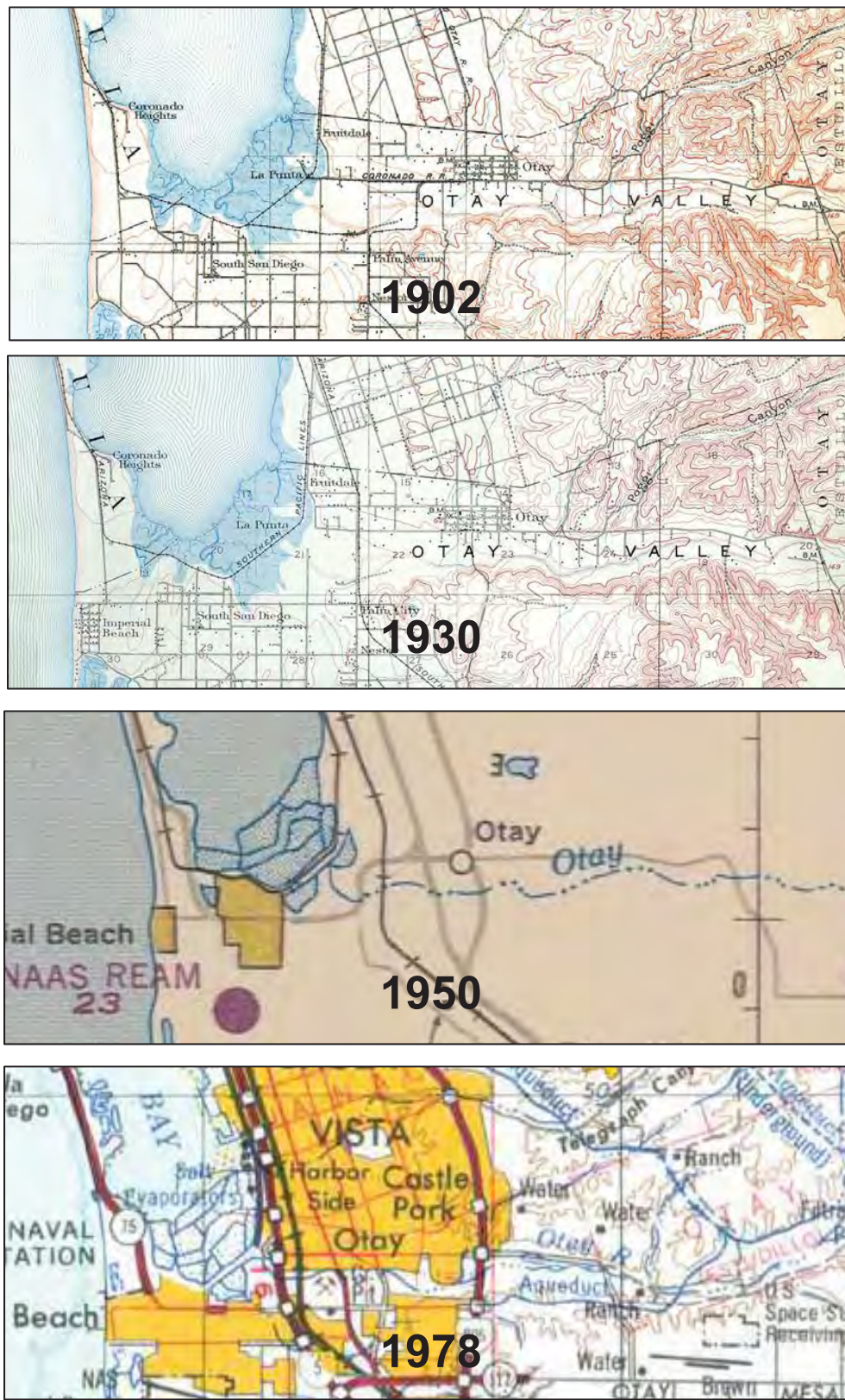
Figure 7.2   Historical Maps of the Otay River near San Diego Bay

Today, essentially all flows and associated sediment from the upper 68% of the watershed are impounded at the Upper and Lower Otay Reservoirs, which are a part of the City of San Diego water supply system. The Upper Otay Reservoir is located at the end of Procter Valley, retaining water and sediment from Proctor Valley Creek. Flows overtopping the upper reservoir are connected to the lower reservoir via a spillway. The Lower Otay Reservoir forms behind Savage Dam, collecting water and sediment from Dulzura Creek, Hollenbeck Canyon, and Jamul Creek. The lower reservoir also receives water from Cottonwood Creek via Morena Dam and Barrett Diversion Dam as well as the Second San Diego Aqueduct.

The Lower Otay Reservoir was originally created in 1897 with water impounded behind a rock-fill structure with a steel core. A series of large storms in January 1916 resulted in failure of the original dam. The catastrophic flood that included loss of life resulted in the destruction of bridges and structures along the canyon and stripping of vegetation and sediment down to bedrock. The flood wave was estimated to be 100-ft high near the dam and 20-ft high in the lower canyon with a peak discharge of 37,400 cfs (USGS 1918), which exceeds the estimated 100-year flood event. The dam was reconstructed between 1917 and 1919 as a gravity-arch dam and renamed Savage Dam, which still stands today.

Savage Dam has reduced sediment delivery by retaining nearly all sediment from the upper watershed resulting in a sediment deficit to the lower river. Sources of fluvial sediments are limited to the 46-mi^2 watershed below the Lower Otay Reservoir. Hence, the sedimentation impact analysis was geographically limited to this region.

Fluvial sediment loading from the Otay River Watershed below the Lower Otay Reservoir was determined from prior studies. Two different methods were analyzed resulting in a range of sediment loadings. The methods were empirically based and account for drainage area and watershed conditions. For Method 1, the sediment loading was determined from estimates for the San Diego River Watershed; and for Method 2, the sediment loading was obtained from a prior watershed loading estimate using a GIS-tool.

7.2.1   Method 1

For Method 1, sediment loading for the Otay River Watershed was estimated by scaling sediment loading estimates for the San Diego River. Given the geographical proximity and similar hydrologic conditions, the sediment loading for the Otay River Watershed was assumed to be proportional to the San Diego River Watershed based on drainage area. The San Diego River is located to the north of Otay River and extends from the Peninsular Ranges to the Pacific Ocean just south of Mission Bay. Similar to the Otay River Watershed, the upper portion of the San Diego River Watershed is controlled by two major reservoirs that cut off flows and sediment to the lower watershed. The San Diego River drainage area covers 432 mi^2 (1,119 km^2), of which 60% is controlled by the El Capitan Dam and San Vicente Dam. The El Capitan Dam was constructed in 1935 and is used by the City of San Diego for municipal uses and

irrigation. The San Vicente Dam was constructed in 1943 and is also used by the City of San Diego for municipal uses.

Sediment loading from the San Diego River was previously estimated by Brownlie and Taylor (1981). Estimates were made using sediment rating curves developed from flow and sediment data collected along the river. As part of the study, annual sediment loads were estimated from 1913 to 1976 for actual and natural conditions, which indicated an average 90% reduction in sediment delivery due to the construction of the El Capitan and San Vicente Dams. Annual sediment loadings for the San Diego River (below the dams) showed a large variability from year to year, thus the average annual loading varied depending on the years selected for the average. In their report, Brownlie and Taylor (1981) provided average sediment loading between two time periods: (i) 1935 and 1956, and (ii) 1943 and 1956. Since the first time period overlaps the implementation of both dams in 1935 and 1943, only the average sediment loading for the second time period is used for this study. For that time period, the estimated average sediment load is1,585 yd^3/yr (1,212 m^3/yr).

Sediment loading for the Otay River Watershed was estimated by scaling the San Diego River sediment loading based on drainage area. A scale factor was calculated as the drainage area of the Otay River below the reservoir to the drainage area of the San Diego River below the reservoir. The resulting estimated sediment loading for the Otay River is 646 yd^3/yr (494 m^3/yr).

7.2.2   Method 2

The second estimate of sediment loadings was obtained from a previous estimate using the Otay Watershed Pollutant Loading (OWPL) Tool (Aspen 2006). The OWPL tool is an Excel spreadsheet setup to estimate annual pollutant loads from the Otay River Watershed and to evaluate best management practices (BMPs). It was developed using PLOAD, a GIS-based tool to calculate runoff volumes and pollutant loads for subwatersheds. The PLOAD calculations employ the EPA Simple Method, an empirical method for estimating pollutant loadings by land use based on drainage area, runoff coefficients, and pollutant event mean concentrations (EMCs). In the OWPL Tool, the Otay River Watershed below the Lower Otay Reservoir is delineated into nine subwatersheds that are individually characterized based on land use and mean annual precipitation. Land uses are used to define runoff coefficients and EMCs based on literature values.

As part of the OWPL tool development, annual loads were estimated for various pollutants including total suspended solids (TSS). The TSS EMCs were obtained from estimates determined by storm water monitoring in Los Angeles County (LACDPW 2000), except for agricultural land use, which was taken from Ackerman and Schiff (2003). Based on the OWPL tool, the annual sediment loading for the Otay River Watershed below the reservoirs was estimated to be 1,360 yd^3/yr (1,040 m^3/yr).

7.2.3   Summary

The estimated sediment loadings based on the above discussed two methods are used to define the likely range of sediment loadings from Otay River Watersheds, i.e. between 646 and 1,360 yd^3/yr. Overall, sediment loadings are relatively small since sediment from the upper portion of the watershed is not transported pass the Lower Otay Reservoir.

7.3   POTENTIAL SEDIMENT DELIVERY FROM OTAY RIVER TO PROPOSED WETLAND

Not all the sediment loadings from the Otay River Watershed will get to the proposed wetland since only portions of the discharge from the Otay River will go through the wetland. In addition depending on the sediment distribution in the sediment loads, some of the larger sediments will deposit along the river bed and only the fine sediments in suspension would be transported with the flow into the proposed wetland.

The total sediment loading generated from the watershed is comprised of eroded sediment of different sizes. Sediment from the Otay River Watershed is generated from areas with roughly half sedimentary and half southern California batholith resulting in a general estimate of sediment composition of 50% fines and 50% sands (Taylor 1981). A portion of the sediment load, primarily sands or gravels, will be primarily deposited within the river bed. Finer sediment material is more likely to stay in suspension and be transported with the river flow. Hence, as a first approximation, it is estimated that only about 50% of the estimated total sediment loadings from the watershed will stay in suspension, i.e. about 323 to 680 yd^3/yr.

Since only a portion of river flow and its associated suspended sediment would flow through the proposed wetland area, the suspended sediment load from the watershed to the proposed wetland area would actually be less than the above estimated 323 to 680 yd^3/yr. During flood events, a portion of the flow overtops the levees along the river and does not flow through the wetland. Based on TUFLOW model results, only about 15, 45 and 60 percent of the flood flow for the 25-, 50- and 100-year flood events will pass through the proposed wetland area. Since sediment loads in general are associated with flood events, based on the model results for the 25-, 50- and 100-year event, it is likely only about 50% of the estimated suspended sediment loads of 323 to 680 yd^3/yr would go through the proposed wetland area, i.e. the annual sediment load to the wetland would be in the range of 160 to 340 yd^3/yr.

7.4   POTENTIAL SEDIMENTATION RATE AT THE PROPOSED WETLAND

It is unlikely that all the suspended sediments passing through the proposed wetland will settle to the bed, but a conservative estimate of the sedimentation rate in the wetland is to assume all the suspended sediment would uniformly deposited over the proposed wetland area (29.62

acres). With this conservative assumption and the estimated annual suspended sediment load of 160 to 340 yd^3/yr, the estimated sedimentation rate in the proposed wetland area would be between 0.04 to 0.08 in/yr (1 to 2 mm/yr). If we assume only about half of the suspended sediment that passes through the proposed wetland would actually settle and stay in the wetland, the average annual sedimentation rate in the wetland would be of the order of 0.02 to 0.04 in/yr (0.5 to 1 mm/yr).

8. EFFECT OF SEA LEVEL RISE

8.1 OVERVIEW

On October 14, 2013, the California Coastal Commission (CCC) released the Draft Sea-Level Rise Policy Guidance for public comment (CCC, 2013). The draft guidance document was prepared by CCC staff to provide a theoretical framework for assessment of sea-level rise in Local Coastal Programs and Coastal Development Permits. The draft guidance policies recognize the science on sea-level rise is constantly evolving, but at the time of the report's publication, the best available science on sea-level rise in California is the 2012 *National Research Council (NRC) Report, Sea-Level Rise for the Coasts of California, Oregon and Washington: Past, Present and Future* (NRC, 2012). The NRC-recommended sea-level rise projections for Southern California (south of Cape Mendocino) are summarized in Table 8.1.

Table 8.1 Potential Sea Level Rise Ranges Using Year 2000 as the Baseline for Southern California (NRC Report 2012)

YEAR	RANGE OF SEA LEVEL RISE (INCHES)
2030	1.6 – 12
2050	5 – 24
2100	16.5 – 66

For this study, the potential effect of sea level rise (SLR) was evaluated for Year 2050 and 2100 using the upper bound of the projected SLR shown in Table 8.1, i.e. 24 inches for Year 2050 and 66 inches for Year 2100. It was assumed that the tide properties remain the same in the future, and SLR effectively simply raise the PMP tide described in Section 4.3.1 uniformly by 24 inches and 66 inches in Year 2050 and 2100, respectively. The resulted PMP tides used as downstream boundary condition for flood modeling for Year 2050 and 2100 are shown in Figure 8.1. In the figure, the timing for the arrival of the 100-year flood from Otay River, Poggi Canyon Creek and Nestor Creek at the upstream boundaries relative to the downstream PMP tides are also shown.
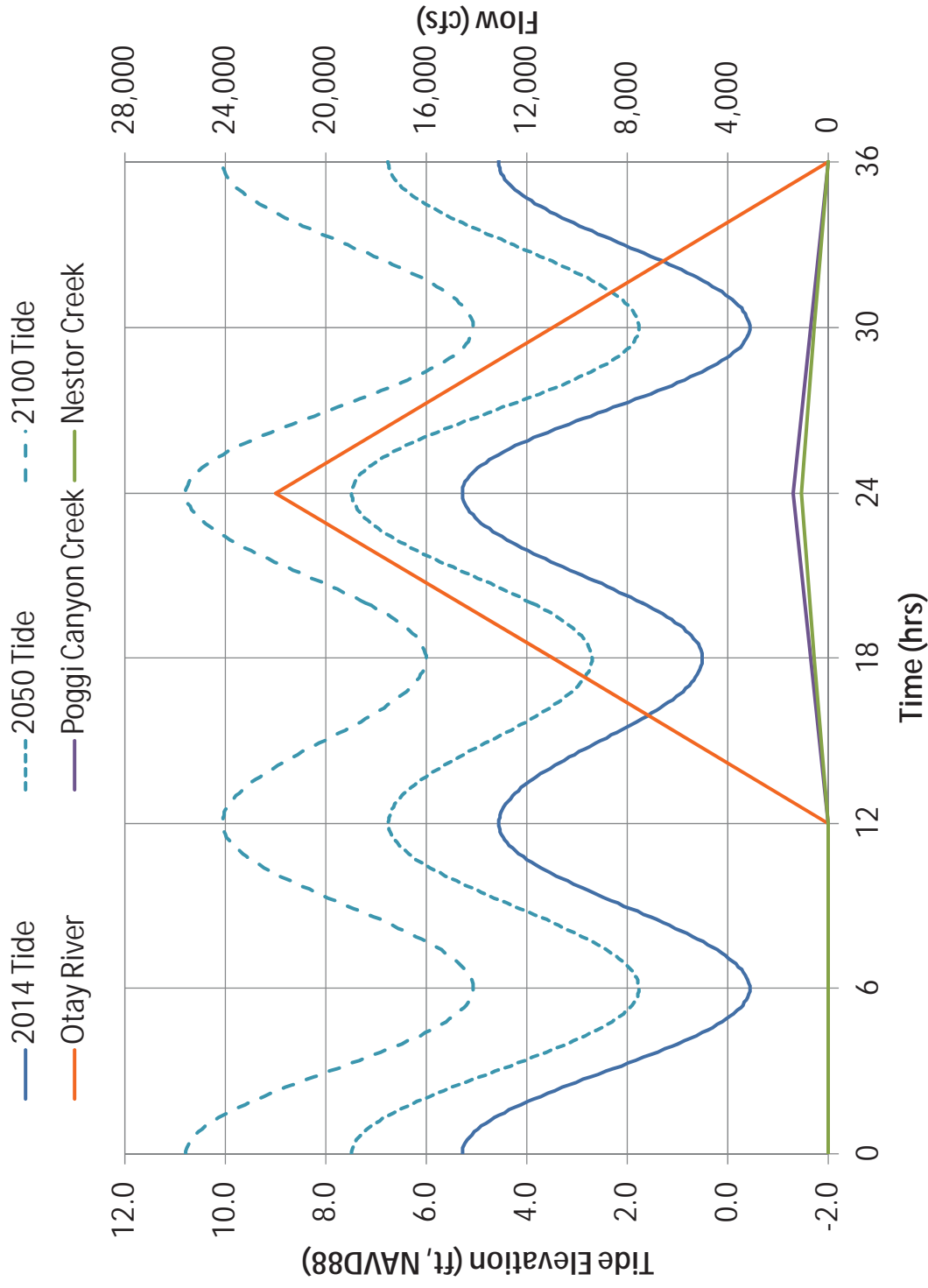
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Figure 8.1   Parametric Mean Periodic Tides in Years 2050 and 2100 and 100-Year Flood Hydrographs

8.2   FLOOD MODELING RESULTS

8.2.1   Existing Conditions

The spatial plots of maximum water elevations for the entire model domain for current, year 2050 and 2100 sea levels under Existing Conditions are shown in Figure 8.2. As shown in the figure, there is no noticeable difference in the extent and flood elevations upstream of I-5 Bridge for Year 2050 and 2100 compared with current sea level condition and minor difference in the ORF. To better illustrate the difference in flood elevations for areas in the ORF, the maximum water elevations for the ORF are shown in Figure 8.3 with a different color scale that allows a better comparison for this area.   As shown in the figure, the only difference in the maximum water elevations in the ORF between Year 2050 and current condition is at the restored Salt Ponds 10 and 11 which are connected to San Diego Bay. With only a 2-ft rise in SLR for Year 2050, the maximum tide water levels are still lower than the levees of the salt ponds; hence there is no change in the flood water elevations during the 100-year flood. However, for Year 2100 with a 5.5 ft rise in SLR, the tide elevations will be higher than the salt pond levees; hence the salt pond will be inundated as illustrated in Figure 8.3.

8.2.2   Flood Impact with Proposed Conditions (Intertidal and Subtidal Alternatives)

The spatial plots of maximum water elevations for the entire model domain for Existing Conditions and with proposed alternatives with Year 2050 SLR are compared in Figure 8.4. With the color scale that can show water elevations for the entire domain, there is no noticeable difference in the extent and flood elevations for the Existing Conditions and proposed alternatives. To better illustrate any potential impact of the proposed alternative to flood levels in the ORF, the maximum water elevations downstream of the I-5 Bridge are compared in Figure 8.5 with a color scale that can show smaller differences. Similar to the results shown in Figure 5.10 for current sea level, the proposed alternatives would result in slightly lower maximum flood water elevations near the proposed wetland areas compared to Existing Conditions. In Year 2050, with a 2-ft SLR, there is still no flooding at Pond 29 under Existing Conditions. However, as expected, since Pond 29 would be flooded under the proposed alternatives with current sea level, there would be increased flooding in Pond 29 in Year 2050.

Similar comparisons of maximum water elevations for Existing Conditions and under proposed alternatives for Year 2100 with 5.5-ft SLR are shown in Figures 8.6 and 8.7. The impacts of the proposed alternatives in 2100 would still be confined to areas near the proposed wetland location. However, with a 5.5-ft SLR in Year 2100, Pond 29 would be inundated even under Existing Conditions.
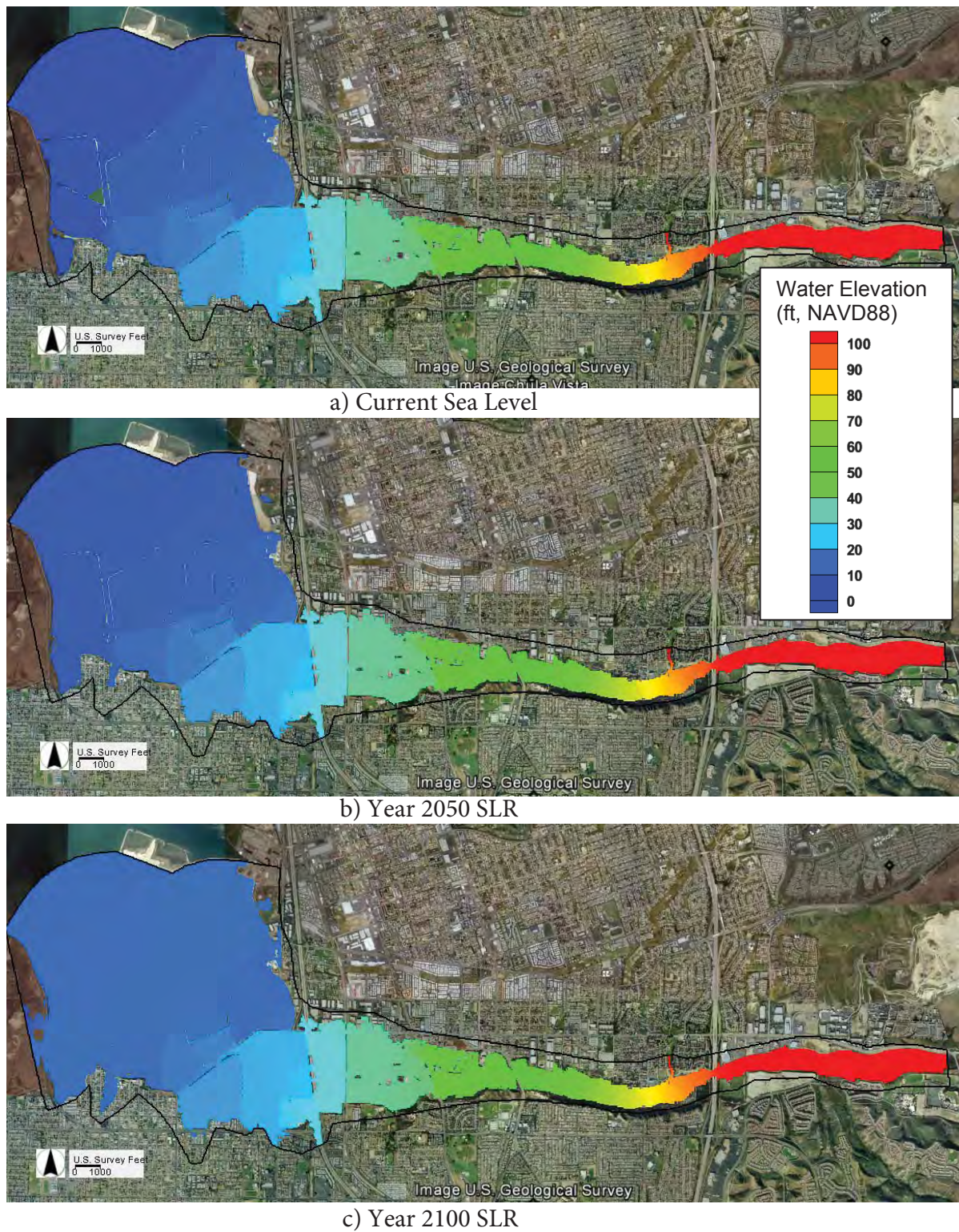
a) Current Sea Level

b) Year 2050 SLR

c) Year 2100 SLR

Figure 8.2   100-Year Flood Maximum Water Elevations under Existing Conditions for Current Sea Level, 2050 and 2100 SLR Scenarios

Water Elevation
(ft, NAVD88)

5 7 9 11 13 15 17 19

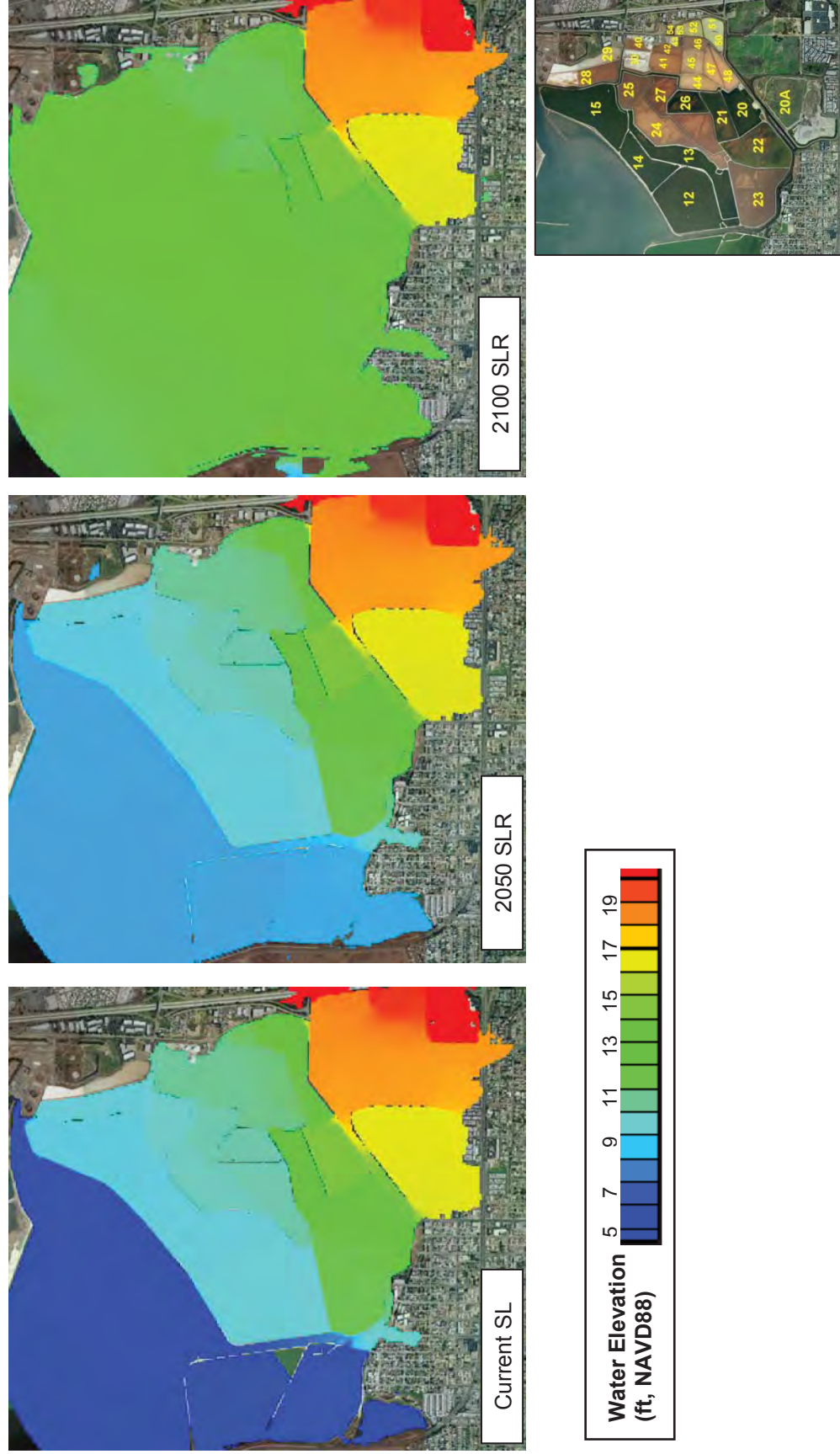**Figure 8.3   100-Year Flood Maximum Water Elevations in Floodplain under Existing Conditions for Current Sea Level, 2050 and 2100 SLR Scenarios**
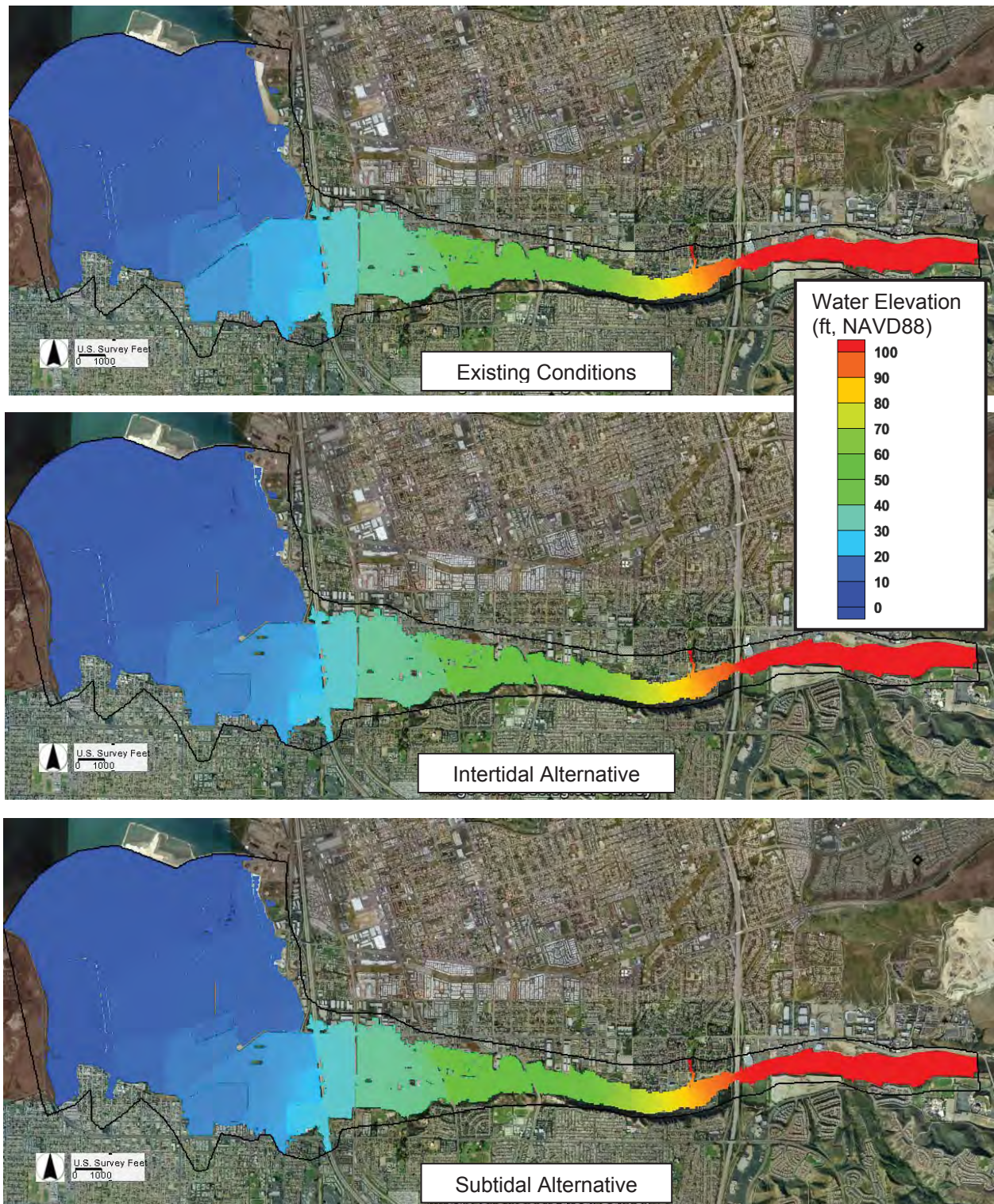
Everest International Consultants, Inc.

Figure 8.4   100-Year Flood Maximum Water Elevations under Existing and Proposed Conditions for 2050 SLR Scenario
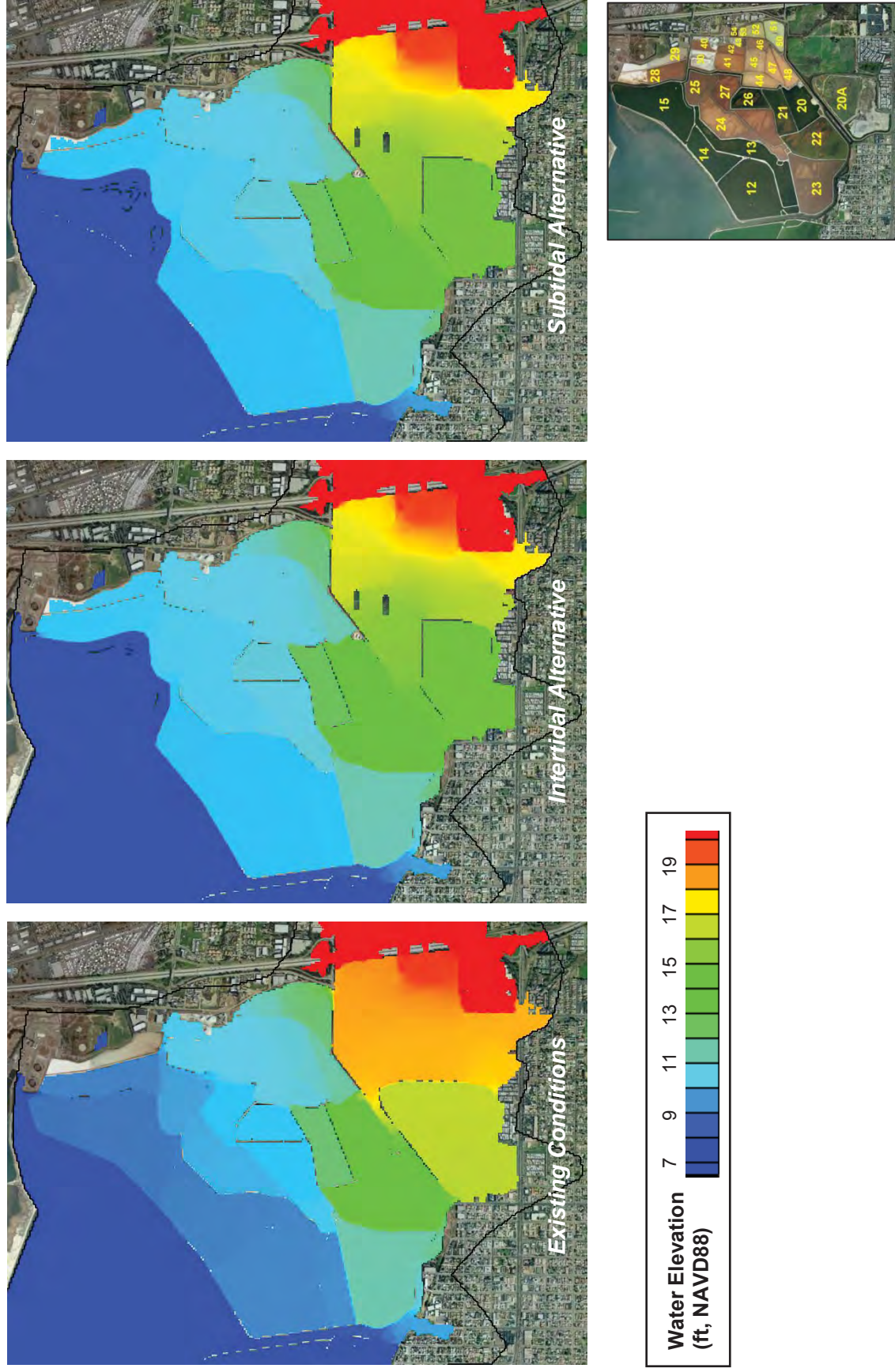
Figure 8.5   Comparison of 100-Year Flood Maximum Water Elevations in Floodplain under 2050 SLR Scenario
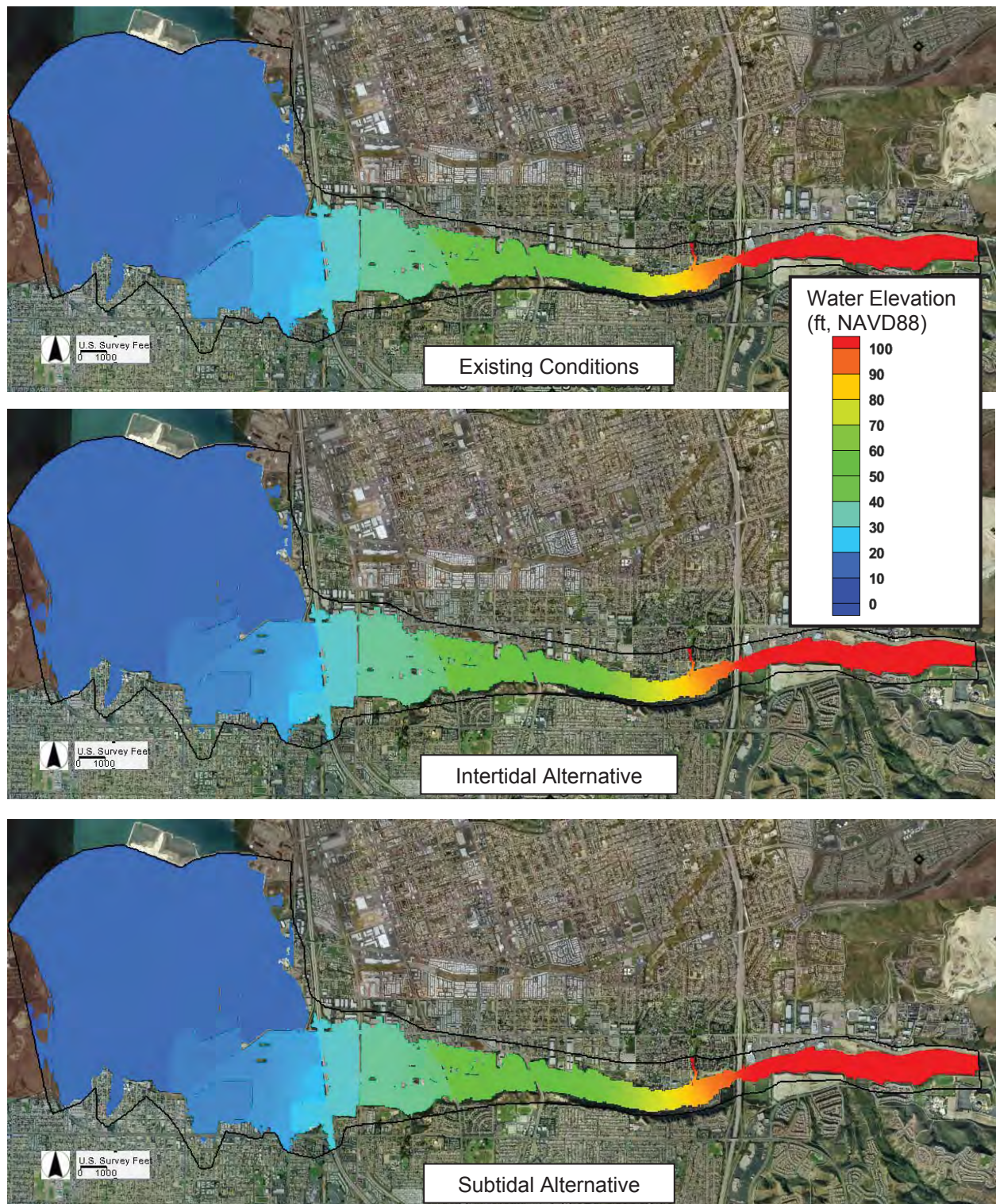
**Figure 8.6   100-Year Flood Maximum Water Elevations under Existing and Proposed
Conditions for 2100 SLR Scenario**

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Subtidal Alternative

Intertidal Alternative

Existing Conditions

Water Elevation (ft, NAVD88)
10 11 12 13 14 15 16 17 18 19 20
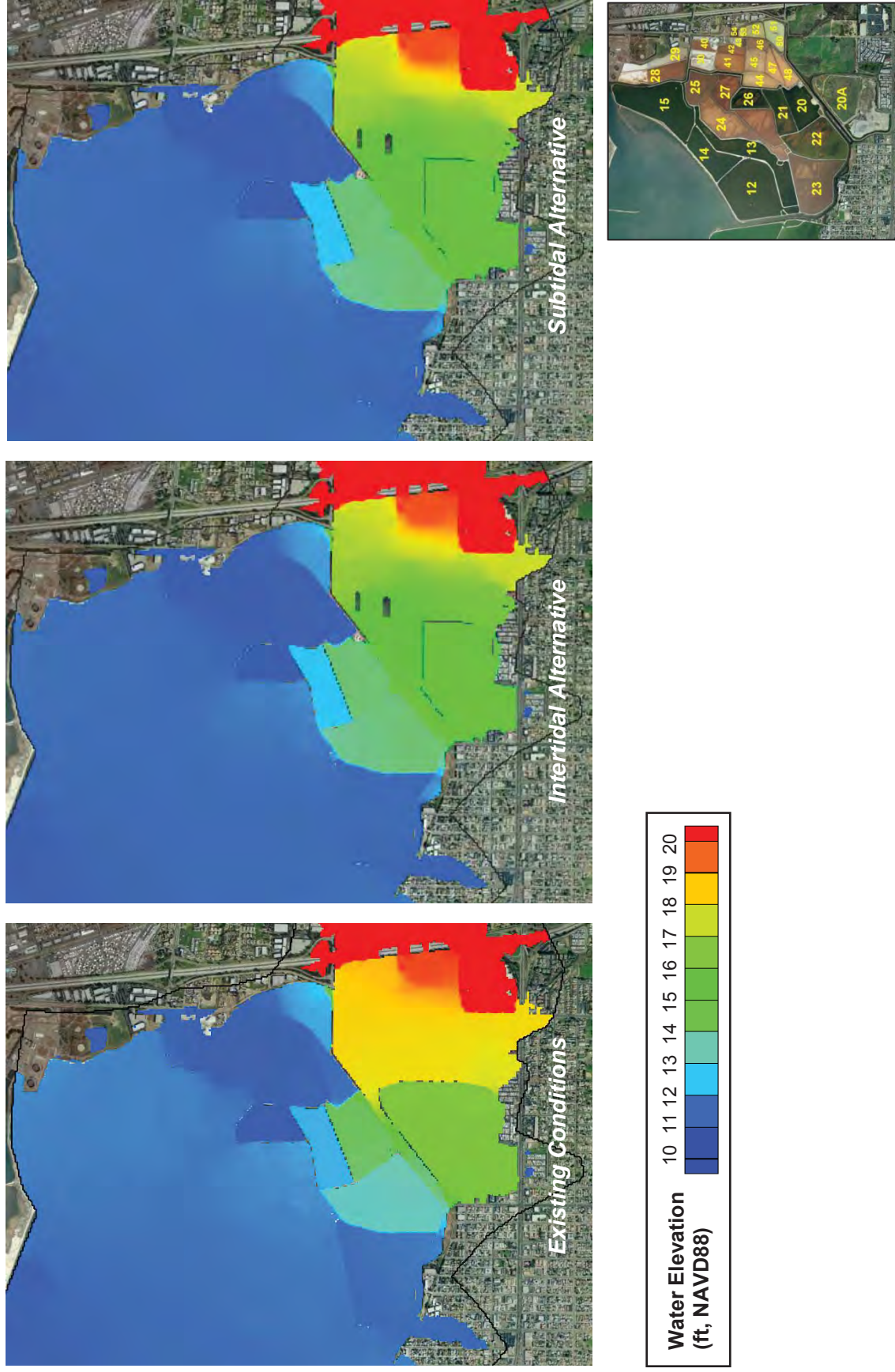
Figure 8.7   Comparison of 100-Year Flood Maximum Water Elevations in Floodplain under 2100 SLR Scenario

Everest International Consultants, Inc.

8.9

9. CONCLUSIONS AND RECOMMENDATIONS

1. Flood impacts of the ORERP are localized and mainly in the vicinity and downstream of the project area (e.g. along the bike paths and in the salt ponds). The ORERP would not have any flood impact to areas upstream of the I-5 Bridge.

2. Along the bike path, for the 100-year flood event, the ORERP would reduce flood elevations at the north end of the bike path adjacent to Pond 48, but increase flood elevations for the south end of the bike path along Pond 22. However, the ORERP would reduce the frequency of flooding along the bike path for smaller flood events. Under existing conditions, flooding would occur along the bike path for flood events with return period of between 10 and 15 years. With the ORERP, flooding along the bike path would not occur up to the 15-year return period flood event.

3. During a 100-year flood event, the ORERP would cause increase in flood elevations at Ponds 12, 13, 14, and 28 compared to Existing Conditions. The increase in flood elevations in Pond 28 would cause overtopping of the levee between Ponds 28 and 29, resulting in flooding of Pond 29 which would not be flooded under Existing Conditions.

4. For most of the ORF, the ORERP would not change flood velocities, including tidally influence areas such as the Western Salt Pond Restoration Project (formerly Ponds 10A, 10, and 11). In general, no erosion impacts are expected within most of the salt ponds. Decreases in erosion impacts were determined for Ponds 20 and Pond 15, as well the central portion of the bike path and river channel adjacent to the proposed wetland.

5. The ORERP would cause higher flows and velocities along the southern portion of the bike path, in the areas between the stock piles, and the levees separated Ponds 12 and 14 from San Diego Bay. The increase in flood velocities may increase local scours in those areas. Additional hydraulic analyses are recommended as part of the final design of the ORERP to determine whether scouring would be a problem. If it is determined that scouring could be a problem for those areas, proper scour protections should be considered in those areas as part of the final design.

6. The potential sedimentation rate at the proposed wetland areas were determined to be low, of the order of 0.02 to 0.04 in/yr (0.5 to 1 mm/yr). The effect of the sedimentation to the wetland is likely to be more than offset by future sea level rise.

7. Under Existing Conditions, the effects of sea level rise (SLR) to flood elevations during a 100-year flood event are confined to the tidally influenced salt pond areas. In Year 2050, with a projection of 2 ft SLR, only Ponds 10A, 10 and 11 (which were recently restored to have tidal connection with San Diego Bay) would have an increase in flood elevations during a 100-year flood compared to current sea level. In Year 2100, with a projected 5.5 ft SLR, the salt ponds would be inundated.

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APPENDIX A   FLUVIAL ANALYSIS AT BAYSIDE PARK, IMPERIAL BEACH

APPENDIX A

FLUVIAL ANALYSIS AT BAYSIDE PARK, IMPERIAL BEACH

A.1.   INTRODUCTION

This appendix summarizes the study conducted for the area in the City of Imperial Beach, where preliminary study results indicated that there would be increase in potential flood impacts as a result of the Otay River Estuary Restoration Project (ORERP). This study area is denoted in yellow in Figure A.1. It is referred to as the Bayside Park area in this appendix. This area is located on the southern bank of the Otay River, near the Salt Pond 23, which is located north of the Otay River. A storm drain constructed under the bikeway connects the study area to the Otay River, as depicted in Figure A.2. The fluvial analysis for the Bayside Park area includes the evaluation of several options to alleviate the flood impacts caused by ORERP. Based on this study and the recommendation of the City staff and the project team members, one of the options has been adopted as part of the proposed feature of ORERP.
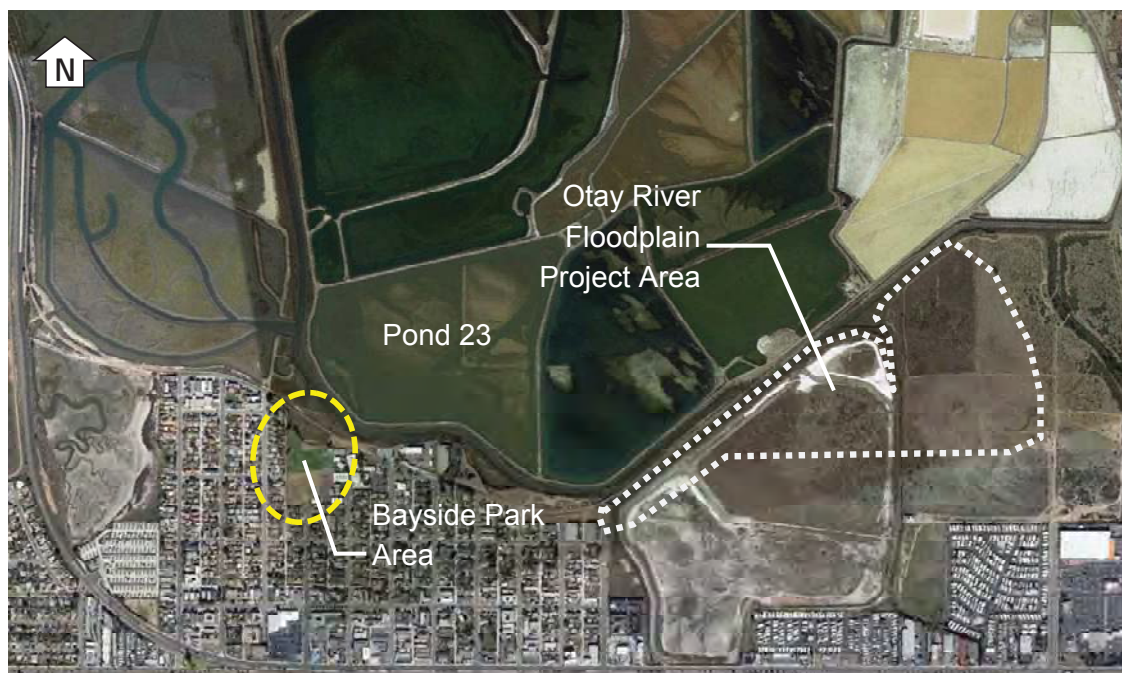

Image: Google Earth Pro

Figure A.1   Bayside Park Study Area

Figure A.2   Photos of Bayside Park Study Area

The fluvial modeling for the Existing Conditions, Intertidal Alternative and Subtidal Alternative are discussed in the main report. The flood impacts of a 100-year storm in the study area were evaluated based on the maximum water elevation results generated from the fluvial model simulations. In all simulations, i.e., the existing conditions and proposed conditions, the results indicate that the area in the vicinity of Bayside Park in the City of Imperial Beach was flooded during a 100-year storm.

The maximum water elevations for the Existing Conditions and for the Subtidal Alternative are presented in Figure A.3 and Figure A.4 respectively. At the location of the Bayside Park area, the maximum water elevation during a 100-year storm under Existing Conditions is 9.2 ft, NAVD88, and that under the Subtidal Alternative is 9.4 ft, NAVD88. The flood elevation result of the Intertidal Alternative is the same as that for Subtidal Alternative at this location. There is an increase of 0.2 ft in flood elevation under the proposed ORERP project conditions.
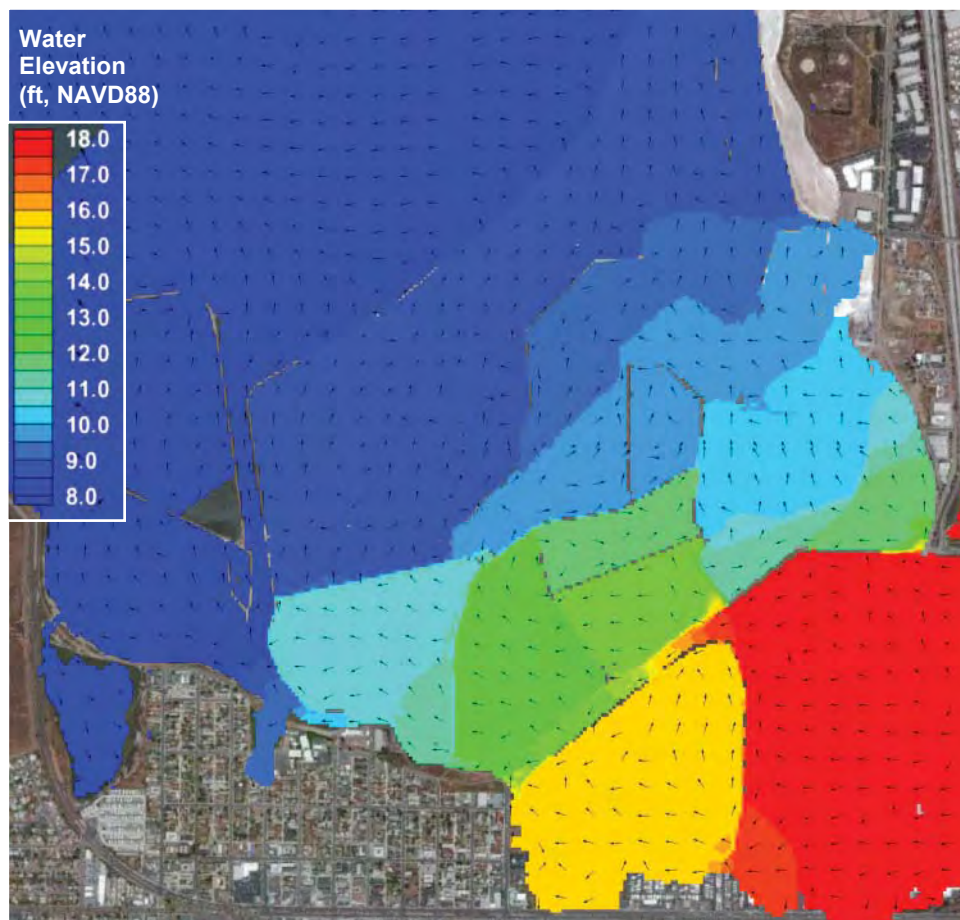

Figure A.3   100-Year Flood Water Elevations for Existing Conditions
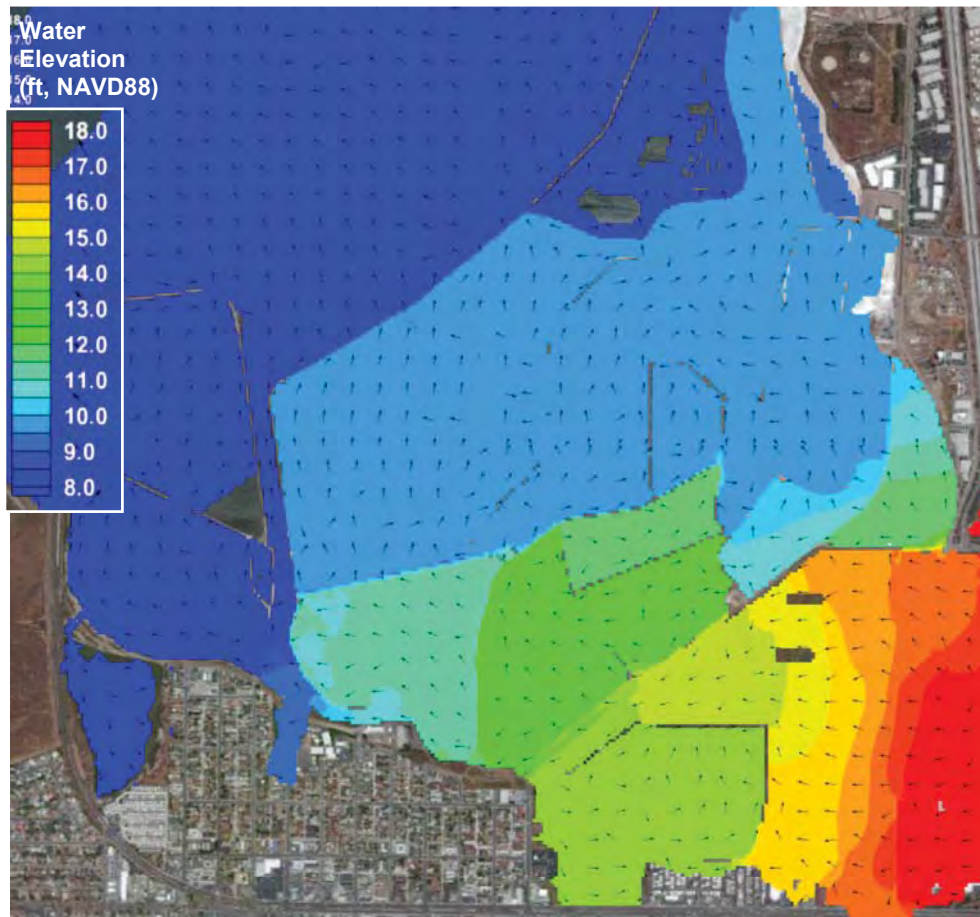
Figure A.4   100-Year Flood Water Elevations for Subtidal Alternative

A.2.   FLOOD IMPACT REDUCTION EVALUATION

Several options were evaluated as potential ways to reduce the increased flood impact in the Bayside Park area. The three options that were evaluated with TUFLOW models are outlined below.

Option 1 – Lower Pond 48 Levee by 4 Feet

Option 1 involves the lowering of the top of the levee on the southern border of Pond 48 (south side of bikeway) from approximately 18 ft, NAVD88 to 14 ft, NAVD88 for a length of 800 feet. The goal of Option 1 is to divert flood flow towards Pond 48 from Otay River such that the flood flow reaching the Bayside Park area would be reduced. Approximately 4,500 CY of material would be removed in this modification. Figure A.5 shows the schematic of the levee modification of Option 1.

Figure A.5   Option 1

Option 2 – Raise Ponds 22 and 23 Levee by 2 Feet

Option 2 involves the raising of the top of the levee between Ponds 22 and 23 by two feet, from an elevation of approximately 11 ft to 13 ft NAVD88 for a length of 1,400 ft. The goal of Option 2 is to divert flood flow away from Pond 23 and Bayside Park neighborhood and towards the northern salt ponds. Approximately 11,500 CY of fill material will be brought in for this modification. Figure A.6 shows the schematic of the levee modification for Option 2.

Figure A.6 Options 2 and 3

Option 3 – Raise Ponds 22 and 23 Levee by 1 Foot

Option 3 is similar to Option 2, the difference is that the levee elevation is increased by one
foot instead of two feet. Under Option 3, the top of the levee between Ponds 22 and 23
would be raised from an elevation of approximately 11 ft to 12 ft NAVD88 for a length of
1,400 ft. The goal of Option 3 is to divert the flood flow away from Pond 23 and Bayside
Park neighborhood. The diverted flow is expected to be less in Option 3 than in Option 2.
Approximately 6,500 CY of fill material will be brought in for this modification. Figure A.6
shows the schematic of the levee modification for Option 3.

A.3.   FLUVIAL MODELING AND RESULTS

The three options were evaluated using the TUFLOW model grids set up for the ORERP
Project. A model simulation for the 100-year storm event was conducted for each of the
three options for the Subtidal and Intertidal Alternatives respectively. The water elevations at
the Bayside Park area were extracted from the model results and are listed in the following

table. The maximum water elevations along the Otay River were plotted and shown in Figure A.7 and A.8 for the Subtidal and Intertidal Alternative respectively. It can be seen that the maximum water elevations in the Bayside Park area is reduced for all the proposed options. Among the three options, Option 2, in which the levee between Ponds 22 and 23 are raised by 2 feet, provides the most flood reduction in the Bayside Park area.

Table A.1   100-Year Flood Maximum Elevations in Bayside Park

SCENARIO	MAXIMUM WATER ELEVATION (FT, NAVD88)
Existing Conditions	9.2
Proposed Conditions *	9.4
Option 1 *	**9.2**
Option 2 *	**9.1**
Option 3 *	**9.2**

* Same results for both Subtidal Alternative and Intertidal Alternative

A.4.   SUMMARY

The results of the analysis were presented to the City of Imperial Beach and other project team members. Based on these results, Option 2 has been selected to be included as one of the proposed features for the Subtidal and Intertidal Alternatives. Option 2 eliminates the project flood impact and reduces the flood elevation of a 100-year storm to 9.1 ft, NAVD88, which is slightly lower than the existing flood elevation of 9.2 ft, NAVD88.
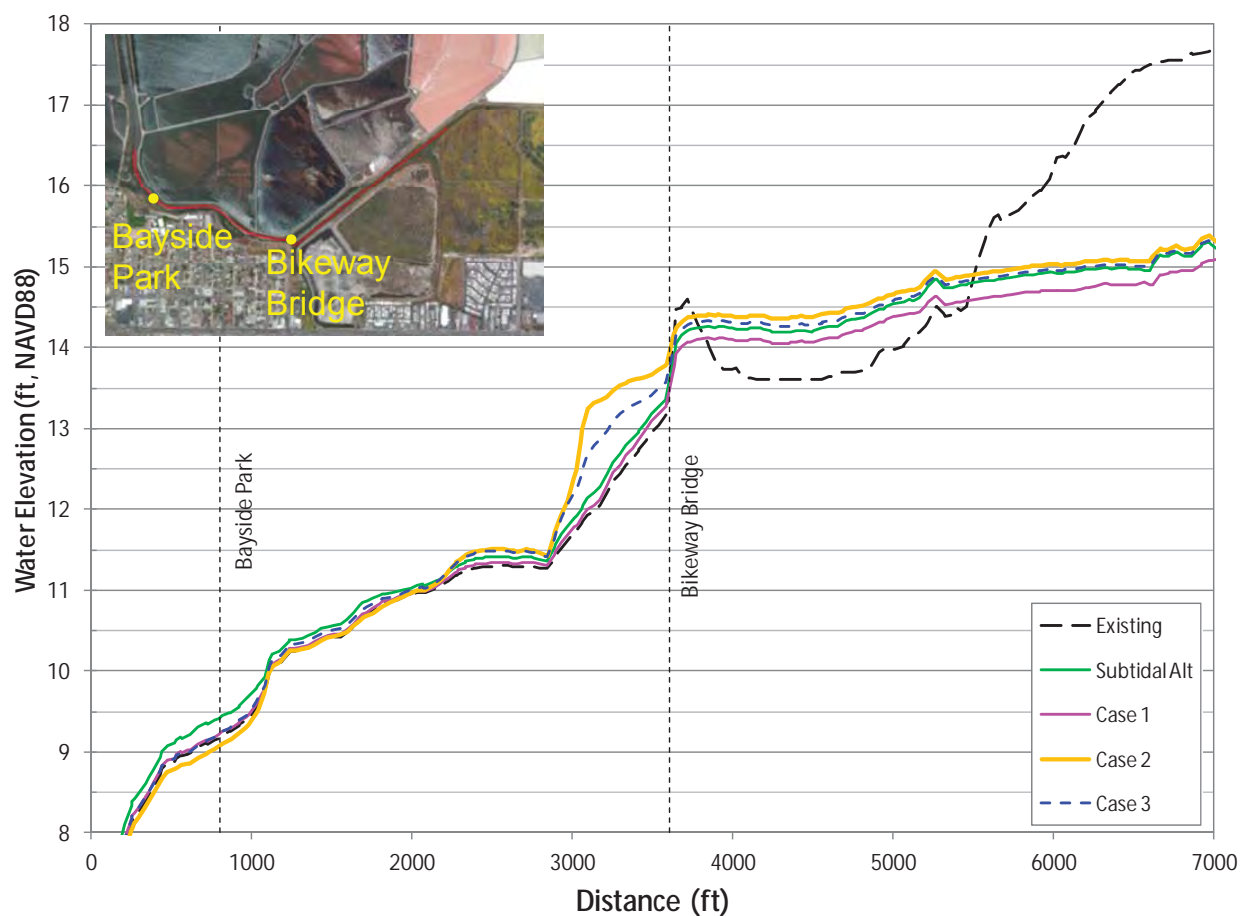
Figure A.7. Subtidal Alternative Maximum Water Elevation Profiles
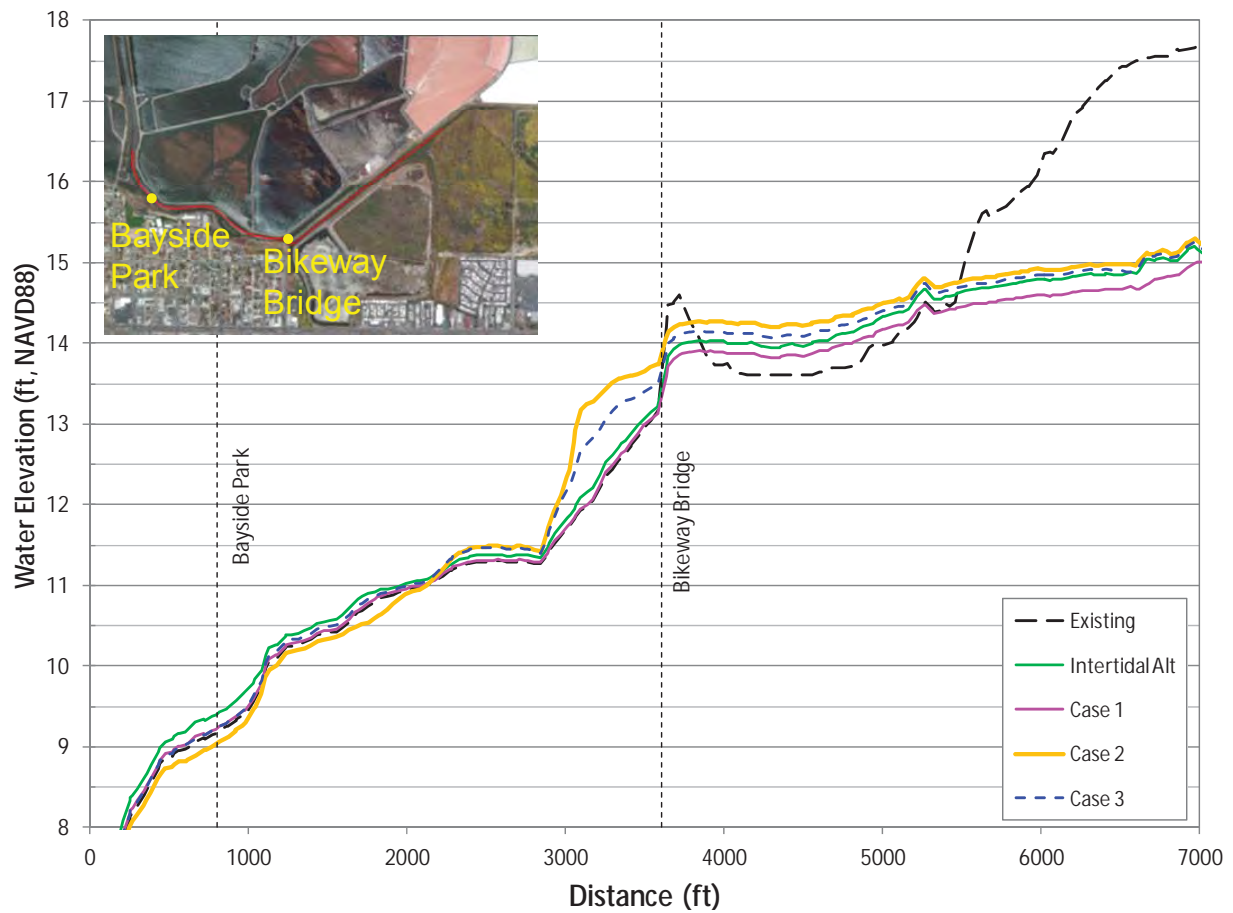
Figure A.8.   Intertidal Alternative Maximum Water Elevation Profiles

APPENDIX B **EROSION PROTECTION FOR THE SOUTH BAYSHORE BIKEWAY BRIDGE**

APPENDIX B

EROSION PROTECTION FOR THE SOUTH BAYSHORE BIKEWAY BRIDGE

B.1.   INTRODUCTION

The TUFLOW model results indicate velocities at the south abutment and channel bed at the South Bayshore Bikeway Bridge (bridge) during the 100-year flood event could be high enough to cause erosion along the slopes of the south abutment and channel bed under the bridge. This appendix provides a summary of a conceptual design to provide scour protection for the south abutment and the channel of the bridge.

B.2.   ABUTMENT PROTECTION

The conceptual design considers the use of rock riprap to protect the south abutment and adjacent bank area of the bridge from scour. The protection design is based on 100-year flood conditions under the Subtidal Alternative, for which TUFLOW model results shows higher velocities along the south abutment of the bridge compared to the velocities under the Intertidal Alternative.

Riprap Size

The conceptual design follows the guidelines of the Hydraulic Engineering Circular No. 23 (HEC-23) published by the Federal Highway Administration (FHWA 2009).   Followed the guidelines for HEC-23, an Isbash equation was used to estimate the riprap size for erosion protection at the south abutment of the bridge

$$\frac{D_{50}}{y} = \frac{K}{(S_S - 1)} \frac{V^2}{gy} \qquad\qquad \text{(1a—for Froude number, Fr} \leq 0.80)$$

$$\frac{D_{50}}{y} = \frac{K}{(S_S - 1)} \left[\frac{V^2}{gy}\right]^{0.14} \qquad\qquad \text{(1b—for Fr} > 0.80)$$

where:

    D_{50} = Median rock diameter, ft
    V = Average velocity at the abutment, ft/s
    S_s = Specific gravity of rock riprap

g   = Gravitational acceleration, 32.2 ft/s^2
y   = Depth of flow at the abutment, ft
K   = Abutment shape coefficient; value depends on whether or not Fr is greater than 0.80, and whether the abutment is of a spill-through or vertical wall shape
Fr = Froude Number based on the velocity and depth adjacent to and upstream of the abutment = $V/(gy)^{1/2}$ where V is the velocity

Parameters used with Equation 1a and 1b are summarized in Table B.1.

Table B.1   Abutment Protection Design Parameters

PARAMETER	SELECTED VALUE	SOURCE/ BASIS FOR SELECTED VALUE
V	8.0 ft/s	TUFLOW model results near the south abutment
S_s	2.65	Average riprap specific weight cited within various literature sources (165 lb/ft^3), and average freshwater density (62.4 lb/ft^3)
y	5.5 ft	TUFLOW model results near the south abutment
Fr	0.60	Calculated from TUFLOW model results
K (Fr ≤ 0.80)	0.89	Eq. 14.1 of HEC-23 for a spill-through abutment
K (Fr > 0.80)	0.61	Eq. 14.2 of HEC-23 for a spill-through abutment

Since the riprap size depends on velocity and water depth, the TUFLOW time series results were examined at multiple locations near the south abutment of the bridge to look for a combined velocity and water depth that may result in the largest rock size for scour protection. The estimated rock size (D_{50}) for scour protection under the 100-year flood is one foot, i.e., Class III riprap.

Riprap Extent

Following the HEC-23 guidance, the riprap apron should extend from the toe of the abutment into the bridge waterway by approximately 25 feet. The downstream riprap coverage should extend back from the abutment by approximately 25 feet as well. The abutment slope should be protected two feet above the expected high water elevation for the design flood, which is higher than low chord of the bridge. Hence, the vertical extent of the abutment riprap is up to the low chord of the bridge.

Riprap Layer Thickness

Following the HEC-23 criteria, assuming the riprap will be placed underwater, the riprap layer thickness should be greater than or equal to the larger of $1.5*D_{100}$ and $2.25*D_{50}$. Given a D_{50} of 1 foot (Class III riprap), maximum allowable D_{100} is 2 ft (based on Table 4.1 of HEC-23). Hence, a riprap thickness of 3 ft is used for the conceptual design.

B.3. CHANNEL BED PROTECTION

Rock riprap will also be used to protect the channel bed from scour. Like the abutment protection design, the channel bed protection design is based on 100-year flood conditions under the Subtidal Alternative.

Riprap Size

Channel bed riprap size was estimated based on fluvial conditions using a design equation in the Hydraulic Engineering Circular No. 11 (HEC-11) published by the Federal Highway Administration (FHWA 1989).

$$D_{50} = 0.001 \frac{C\,(V_a{}^3)}{(d_{avg}{}^{0.5} K_1{}^{1.5})} \qquad\qquad (2)$$

where:

D_{50} = Median riprap particle size

C = Correction factor, $C_{sg}*C_{sf}$

C_{sg} = Correction factor for the specific gravity (S_s) of the rock riprap, $\frac{2.12}{(S_s-1)^{1.5}}$

C_{sf} = Correction factor for the stability factor (SF) to be applied, $\left(\frac{SF}{1.2}\right)^{1.5}$

V_a = Average velocity in the main channel, ft/s

d_{avg} = Average flow depth in the main flow channel, ft

K_1 = Side slope correction factor, $\left[1 - \left(\frac{sin^2\theta}{sin^2\varphi}\right)\right]^{0.5}$

θ = Bank angle with the horizontal

φ = Riprap material's angle of repose

Parameters used with Equation 2 are summarized in Table B.2 below.

Table B.2   Channel Bed Protection Design Parameters

PARAMETER	SELECTED VALUE	SOURCE/ BASIS FOR SELECTED VALUE
S_s	2.65	Average riprap specific weight cited within various literature sources (165 lb/ft^3), and average freshwater density (62.4 lb/ft^3)
SF	1.7	Guidelines for the selection of stability factors, included as Table 1 in HEC-11
V_a	8.0 ft/s	TUFLOW model results under the bridge
d_{avg}	4.6 ft	TUFLOW model results under the bridge
θ	14.8°	Average bank slope at typical channel cross section
φ	40°	Average value cited within various literature sources

Since riprap size depends on velocity and water depth, the TUFLOW time series results were examined at multiple locations near the bridge to look for velocity and water depth sets that may result in larger rock size. The average maximum velocity under the bridge and corresponding average water depth values were selected. A conservative stability factor (SF) of 1.7 was selected to account for the effects of the sharp channel bend at the bridge. As such, the estimated rock size for 100-year flood conditions under the Subtidal Alternative is a D_{50} of 0.5 feet.

B.4.   RECOMMENDED PROTECTION DESIGN

The estimated abutment riprap diameter (D_{50}) is 1 foot, which is twice that of the estimated riprap diameter of 0.5 feet for the channel bed. Since the abutment riprap needs to be extended 25 ft into the channel, it is easier for construction to extend it all across the channel bed instead of having part of the channel protection using the smaller 0.5 ft riprap size. Usage of a uniform riprap diameter and layer thickness for the overall abutment and channel bed protection is recommended for practical construction purposes. Since the underlying soils are fine-grained, a geotextile filter and granular filter should be placed in between the riprap and the soil surface. The recommended conceptual design for the overall revetment protection, including riprap extent, is presented in Figure B.1.
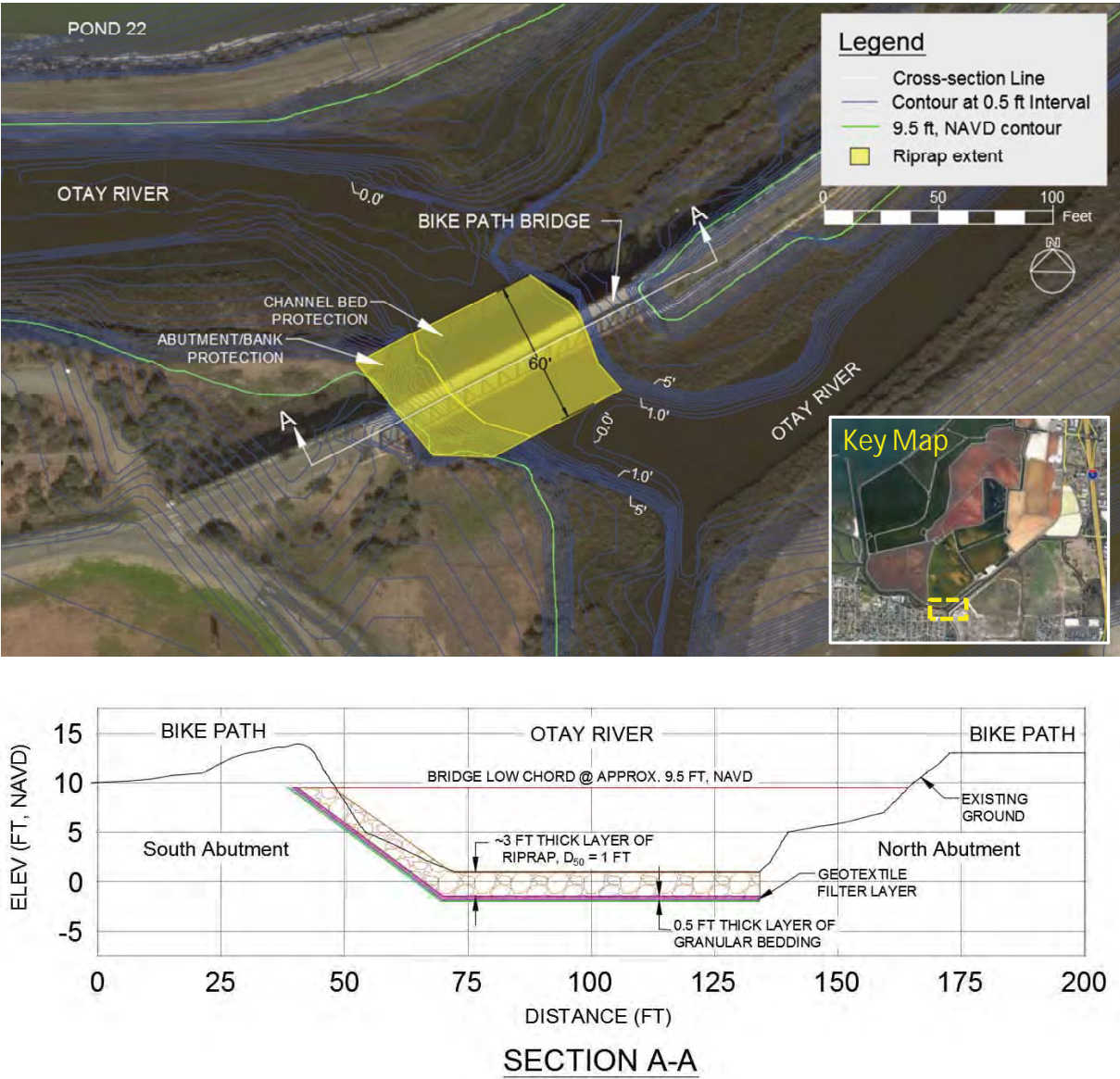
Figure B.1   Conceptual Design for Revetment Protection at the South Bayshore Bikeway Bridge (Do Not Use for Construction)

B.5. REFERENCES

FHWA 1989. Hydraulic Engineering Circular No. 11 (HEC-11). Design of Riprap Revetment. Publication No. FHWA-IP-89-016. Federal Highway Administration, U.S. Department of Transportation. March 1989.

FHWA 2009. Hydraulic Engineering Circular No. 23 (HEC-23). Bridge Scour and Stream Instability Countermeasures, Third Edition.   Publication No. FHWA-NHI-09-112. Federal Highway Administration, U.S. Department of Transportation. September 2009.